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Coastal and Hydraulics Laboratory

US Army Corps of Engineers®
Engineer Research and Development Center

Monitoring Completed Navigation Projects Program

Effects of Breakwater Construction on Tedious Creek Small Craft Harbor and Estuary, Maryland

Barbara P. Donnell, Michael J. Briggs, Zeki Demirbilek, Thad C. Pratt, Michael W. Tubman, Robert D. Carver, and Karen M. Nook

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Final report

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Under MCNP Work Unit 11M17

Abstract: Tedious Creek is a small, funnel-shaped estuary located on the eastern shoreline of the Chesapeake Bay in Dorchester County, MD. Prior to the construction of the breakwater in 1997, the orientation of Tedious Creek allowed the transmission of storm waves that, at times, caused substantial damage to local vessels. The breakwater differed in geometry from the plans tested in 1994. Foundation problems encountered in the field resulted in a shortening of the breakwater, and a wider opening between the two breakwater sections resulted. Local watermen complained that the breakwater was not providing the authorized level of protection at the county boat dock and public piers on the south shore. It was suggested that the as-built 122-m (400-ft) gap opening should be reduced to the authorized 91-m (300-ft) gap opening.

The objective of the Monitoring Completed Navigation Projects study was to determine if the harbor and its structures were performing (both functionally and structurally) as predicted by model studies used in the project design. Specific field data would be obtained and analyzed. These data were used in numerical simulation modeling to ascertain the level of wave protection provided by the as-built breakwater structure, and to compare this level of protection to that which would have been provided if the authorized structure had been built. A third hypothetical structure with a 61-m (200-ft) gap opening also was evaluated. No adverse environmental effects such as breakwater deterioration, shoreline erosion, or scour near the breakwater could be ascertained by analyses of these data.

Wave height transformations were performed with varying wave heights, tides, storm surge levels, and incident wave angles using numerical models STWAVE (no diffraction), STB3 (diffraction), and CGWAVE (refraction, diffraction, and energy losses). Evaluations were performed for (a) storm waves, as-built and authorized structures, (b) moderate waves, as-built and authorized structures, (c) typical daily waves, as-built and authorized structures, and (d) storm and typical daily waves, hypothetical structure. For all wave conditions, any reduction in wave heights at the county boat dock and public piers by reducing the gap opening in the breakwater from the as-built to the authorized opening would be minimal and insignificant. Reduction of the gap to the hypothetical 61-m (200-ft) opening resulted in about a 10-percent reduction in typical daily condition (considered insignificant) and modification of the structure to this degree (from as-built 122-m (400-ft) gap to hypothetical 61-m (200-ft)) would not be justified.

The functionality of circulation and flushing of the as-built condition was evaluated by applying two models (RMA2 and RMA4) within the TABS-MD suite of numerical models. RMA2 was used to demonstrate general hydrodynamic circulation patterns resulting from verification of the August 2001 field data. RMA4 was used to analyze harbor flushing. The as-built condition appears to maintain good harbor circulation, with velocities below any threat to boats that frequent the harbor. RMA4 flushing tests indicate that the harbor has adequate flushing, and compares favorably to the no-structure flushing test.

Moreover, field data and observations made by the Baltimore District during project location site visits indicate that wave conditions preventing satisfactory operations at the county boat dock facility often result from northwesterly waves generated locally on Tedious Creek, rather than by waves propagating through the breakwater gap from a northeasterly direction.

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Preface

The studies reported herein were conducted as part of the Monitoring Completed Navigation Projects (MCNP) Program (formerly Monitoring Completed Coastal Projects Program. Work was conducted under MCNP Work Unit No. 11M17, "Tedious Creek, Maryland." Overall program management of the MCNP is provided by Headquarters, U.S. Army Corps of Engineers (HQUSACE). The Coastal and Hydraulics Laboratory (CHL), U.S. Army Engineer Research and Development Center (ERDC), Vicksburg, MS, is responsible for technical and data management, and support for HQUSACE review and technology transfer. Program monitors for the MCNP Program are Barry W. Holliday, HQUSACE, and Charles B. Chesnutt, U.S. Army Institute for Water Resources. MCNP program managers during the conduct of this study were Robert R. Bottin, Jr., and Dr. Lyndell Z. Hales, CHL.

This research was conducted during the time period October 2000–September 2004 under the general supervision of Thomas W. Richardson, Director, CHL; Dr. Rose M. Kress, Chief, Navigation Division (ND), CHL; and Bruce A. Ebersole, Chief, Flood and Storm Protection Division (FSPD), CHL; and under direct supervision of Dennis G. Markle, former Chief, Harbors, Entrances, and Structures Branch (HESB), ND, CHL; Jose E. Sanchez, present Chief, HESB; David R. Richards, former Chief, Hydrologic Systems Branch (HSB), FSPD, CHL, Earl V. Edris, Jr., current Chief, HSB; and William Birkemeier, Chief, Field Data Collection and Analysis Branch (FDCAB), FSPD, CHL.

The studies reported herein were conducted by Drs. Michael J. Briggs and Zeki Demirbilek, and Robert D. Carver (retired), HESB; and Barbara P. Donnell, HSB, and Thad C. Pratt and Michael W. Tubman, FDCAB. MCNP District Team Member Karen M. Nook, U.S. Army Engineer District, Baltimore, Engineering Division, contributed significantly to the development and execution of this study. MCNP Principal Investigator was Ms. Donnell.

COL Richard B. Jenkins was Commander and Executive Director of ERDC. Dr. James R. Houston was Director.

Unit Conversion Factors

Multiply	By	To Obtain
feet	0.3048	meters

1 Introduction

Monitoring Completed Navigation Projects program

The goal of the Monitoring Completed Navigation Projects (MCNP) program (formerly the Monitoring Completed Coastal Projects program) is the advancement of coastal and hydraulic engineering technology. The program is designed to determine how well projects are accomplishing their purposes and how well they are resisting attacks by their physical environment. These determinations, combined with concepts and understanding already available, will lead to (a) the creation of more accurate and economical engineering solutions to coastal and hydraulic problems, (b) strengthening and improving design criteria and methodology, (c) improving construction practices and cost effectiveness, and (d) improving operation and maintenance techniques. Additionally, the monitoring program will identify where current technology is inadequate or where additional research is required.

To develop direction for the program, the U.S. Army Corps of Engineers (USACE) established an ad hoc committee of engineers and scientists. The committee formulated the objectives of the program, developed its operation philosophy, recommended funding levels, and established criteria and procedures for project selection. A significant result of their efforts was a prioritized listing of problem areas to be addressed. This is essentially a listing of the areas of interest of the program.

Corps offices are invited to nominate projects for inclusion in the monitoring program as funds become available. The MCNP program is governed by Engineer Regulation 1110-2-8151 (Headquarters, U.S. Army Corps of Engineers (HQUSACE) 1997). A selection committee reviews and prioritizes the nominated projects based on criteria established in the regulation. The prioritized list is reviewed by the Program Monitors at HQUSACE. Final selection is based on this prioritized list, national priorities, and availability of funding.

The overall monitoring program is under the management of the Coastal and Hydraulics Laboratory (CHL), U.S. Army Engineer Research and Development Center (ERDC), with guidance from HQUSACE. An individual monitoring project is a cooperative effort between the

submitting District and/or Division office and CHL. Development of monitoring plans and conduct of data collection and analyses are dependent upon the combined resources of CHL and the District and/or Division.

Project location

Tedious Creek is a small, funnel-shaped estuary located on the eastern shoreline of the Chesapeake Bay in Dorchester County, MD (Figures 1 and 2). This estuary is located in an area that provides excellent access to many productive fishing grounds in Chesapeake Bay. Tedious Creek Harbor provides anchorage to over 100 vessels involved in commercial and/or recreational fishing.

The orientation of Tedious Creek allows the transmission of storm waves that, at times prior to the construction of the breakwater in 1997, caused substantial damage to local vessels. A Section 107 Feasibility Report and Integrated Environmental Assessment (U.S. Army Engineer District, Baltimore 1995) documents how a constructed breakwater design could provide a cost-effective means of minimizing that damage. It also indicates that adverse environmental impacts from a constructed breakwater would be minor and potentially offset by the creation of additional marsh habitat adjacent to the project.

Although the environmental impacts caused by the breakwater project were, and are still, thought to be minor, the project area is undergoing large-scale hydrologic and environmental changes for other reasons. The general area was once dominated by large freshwater marshes that supported a variety of plant and animal species and, in particular, large concentrations of waterfowl. Blackwater Wildlife Refuge, just north of Tedious Creek, is an excellent example of the wetlands that the Chesapeake Bay region depends on to support its economy and way of life. The marshes of Blackwater Wildlife Refuge are experiencing a serious decline for a variety reasons including, but not limited to, sea level rise, salinity intrusion, wave attack, and nutria infestation.

With these issues in mind, this MCNP research investigation will be conducted in sufficient detail to ascertain whether observed impacts are project caused, or are a part of an overall hydrologic decline.

Figure 1. Location of Tedious Creek estuary in the Chesapeake Bay, MD.

Figure 2. Orientation of Tedious Creek estuary, Chesapeake Bay, MD.

Much of the landscape in the vicinity of Tedious Creek is dominated by brackish (mesohaline) tidal marsh (Figure 3). Tidal wetlands support the estuarine food chain, serve as nurseries for commercial fisheries, provide shore protection for the mainland, and help maintain water quality. Because of their valued ecological functions, tidal wetlands of the Chesapeake Bay are given high priority for protection. Dominant marsh species at Tedious Creek include smooth cordgrass, salt hay, spikegrass, big cordgrass, reed grass, and needlerush. Wetlands occur at both the northern and southern shores of the Tedious Creek estuary.

Figure 3. Tidal marsh wetlands, Tedious Creek estuary.

Submerged aquatic vegetation (SAV) beds occur near the mouth of Tedious Creek. SAV beds provide spawning, nursery, feeding, and refuge habitat for numerous species of waterfowl, finfish, and shellfish, and affect nutrient cycling, sediment stability, and water turbidity. SAV abundance appears to rank among its lowest levels in recorded history within the Chesapeake Bay. Because of their valued ecological functions, and because of their relatively depleted abundance, SAV beds are given high priority for protection and eventual restoration.

Problem

Prior to the construction of the breakwater at the entrance to Tedious Creek estuary in 1997, the major problem affecting navigation at Tedious Creek was damage caused by storm waves entering the estuary. Due to the orientation of Tedious Creek to Fishing Bay, storm waves from the northeast, east, and southeast entered the harbor unobstructed, sometimes resulting in significant damage to vessels, operations, and facilities. During storm events, waves caused the vessels to impact pilings, causing various levels of damage, and sometimes resulting in sinking of a vessel. In addition, storm waves resulted in delays to navigation during attempts to dock a vessel, and were estimated to affect watermen approximately 75 days per year. Storm waves in Tedious Creek also had an economic impact on the crab shedding industry. During storm events, silt-laden water was inadvertently pumped into crab shedding tanks, causing suffocation of peeler crabs. Similarly, damage to floats and tanks was also reported.

Several plans of improvement were examined at various structure heights for providing protection at Tedious Creek. Various structural and nonstructural solutions were evaluated, including breakwaters, bulkheads, and vessel relocation. Various breakwater layouts were selected for further detailed evaluation. The plan selected for construction was a breakwater in two sections each 245 m (800 ft) long, with one section connected to the north shore and one section connected to the south shore. Each section had a 15-m (50-ft) gap about midpoint location of both sections to improve water circulation in the harbor. There would be a central navigation gap 91 m (300 ft) wide located between the north and south sections of the breakwater.

In addition to favorable economic return, the proposed breakwater had the potential to offer another beneficial purpose with the creation of marsh habitat adjacent to the existing Sandy Point shoreline using material excavated from the foundation of the south section. There were minimal expected impacts to water circulation and environmental resources associated with the breakwater construction in 1997.

Predicted breakwater project performance

The performance of the breakwater project was predicted by using both analytical and numerical models (U.S. Army Engineer District, Baltimore

1995), which cites a diffraction diagram analysis from the *Shore Protection Manual* (U.S. Army Corps of Engineers 1984) as being used to determine the gap width and alignments of the breakwater sections. The design of breakwater crest heights was analyzed using the "Wave Transmission Through Permeable Structures" application of the ACES program. The 1.8-m (6-ft) crest height was selected based on an economic benefit analysis that maximized the project's net benefits.

Hydrodynamic model investigations conducted in 1994 by the Baltimore District and CHL were used to determine if there would be adverse navigation or environmental effects from the breakwater. The RMA2 hydrodynamic model within the TABS-MD suite of two-dimensional, vertically averaged numerical models, which is included in the Surface Water Modeling System (SMS), was used to study impacts throughout Tedious Creek and Fishing Bay. The model predicted tide heights and current velocities for the existing and two plan conditions. General observations were that currents would increase in the throats of the breakwaters, and they would decrease along the middle of the breakwaters. A qualitative assessment of tidal sedimentation processes based on the tidal circulation modeling suggested that channel maintenance may be improved in the channels, and that sediment may accumulate near the breakwaters.

It is important to note that the breakwater constructed in 1997 differed in geometry from the plans tested in 1994. Foundation problems encountered in the field resulted in a shortening of the breakwater by about 30.5 m (100 ft) and, therefore, a wider opening between the two breakwater sections. As a result, it was anticipated that the level of wave protection provided would be different from the simulated plan. The same is true for tidal circulation impacts and their inferred sedimentation patterns. For consistency, the same models should have been used to simulate the as-built structure performance as had previously been used in the original simulations of the other structural alternatives for tidal heights and current velocities. The TABS-MD numerical hydrodynamic model (RMA2) and the flushing analysis model (RMA4) as applied to the as-built harbor are presented in Chapter 5.

Observed breakwater project performance

After completion of the Tedious Creek breakwater project, local watermen complained that the breakwater was not providing the authorized level of

protection at the county boat dock and public piers on the south shore (Figures 4 – 6). It was suggested that the as-built 122-m (400-ft) gap opening should be reduced to the authorized 91-m (300-ft) gap opening. The Baltimore District requested that Offshore and Coastal Technologies Incorporated—East Coast develop, calibrate, and verify a wave model for that area. Three different breakwater gap scenarios were modeled: (a) the authorized 91-m (300-ft) gap, (b) the as-built 122-m (400-ft) gap, and (c) a theoretical 61-m (200-ft) gap (Offshore and Coastal Technologies Inc. 2001).

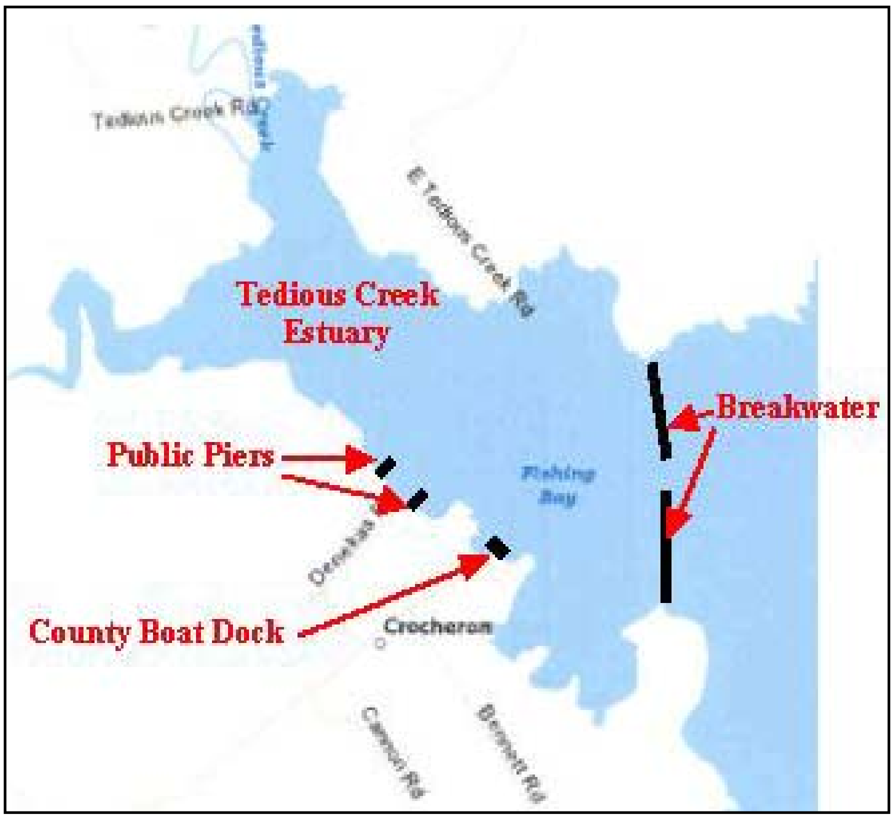

Figure 4. Location of Tedious Creek breakwater, and public piers and county boat dock on the south shore subjected to excessive wave heights during storm conditions.

Figure 5. County boat dock and breakwater section on the horizon (highlighted in red).

Figure 6. South tip of northern section of Tedious Creek breakwater at navigation gap.

The Offshore and Coastal Technologies study determined that the as-built gap reduced normal daily wave heights at the county boat dock to less than 0.2 m (0.5 ft), which is considered a tolerable level for vessel unloading and mooring. Thus, an additional 10-percent reduction by closing the gap to 91 m (300 ft) would be considered insignificant and modification of the project would not be justified.

More significantly, field data and observations made during project location site visits indicated that wave conditions preventing satisfactory operations at the county boat dock facility often result from northwesterly waves generated locally on Tedious Creek, rather than by waves propagating through the breakwater gap from a northeasterly direction (Nook 2002).

At about this time, the Baltimore District nominated the Tedious Creek breakwater project for monitoring and evaluation as part of the MCNP program. This nomination was subsequently authorized for funding by HQUSACE, and the MCNP study was conducted during the time period 2001 through 2004.

MCNP monitoring plan

The objective of the monitoring plan was to determine if the harbor and its structures were performing (both functionally and structurally) as predicted by model studies used in the project design. Wave, current, sediment, and bathymetry measurements at the project site would determine the effectiveness of the functional design aspects. The structural aspects would be investigated using ground-based surveys and airborne photogrammetry.

Bathymetry data

Bathymetry data would be collected for the entire wetted area inside and just outside of the Tedious Creek area. The purpose was to provide an accurate baseline of bathymetric conditions now, and then again in the future, to determine the effects of the breakwater on bathymetry. Possible bathymetric changes could include the accumulation of sediment to the lee of the breakwaters, scour in the opening between the breakwaters, and erosion of marshes adjacent to the breakwaters. Marsh formation within the protected harbor was also possible.

Ground-based survey

A limited ground-based survey would be conducted to establish control (monuments and targets) for photogrammetric analysis of the structures. Also, a walking inspection would be conducted to document any broken armor units or dislodged stones.

Photogrammetry

A variety of photogrammetric and surveying methods were planned to document changes in Tedious Creek. Changes that might be expected include movement of the breakwater, shoreline/wetland migration, and sediment redistribution. Methods would be chosen based on their ability to provide the needed accuracy for reasonable cost.

Methods used historically to document breakwater movement have used stereoscopic imagery to determine base versus plan differences in elevation. This is particularly useful in breakwaters where much armor unit movement and/or breakage of individual armor units is expected. In these cases, monuments on individual rocks are less useful since they experience substantial movement. In cases where the stone size is conservatively large for the expected waves (as in Tedious Creek), it is possible to use recent advances in surveying methods, specifically the differential global positioning system (DGPS), to set up numerous monuments on individual units in the breakwater to document three-dimensional movements. The expected accuracy of such methods (<0.03 m (0.1 ft) vertically) is arguably greater than even the best stereoscopic analysis.

Both methods would be used in the initial year. In the second year, breakwater movement would be detemined using (a) surveying and (b) digital photographic methods for the purpose of selecting the methods to be used in the final year. Both methods might be used in the final year. Perhaps the most compelling reason for using both methods in the final year is that photogrammetry provides a side benefit of documenting wave directions, shoreline changes, and possibly sedimentation patterns seen as plumes in the water.

Wave data

An ongoing Tedious Creek wave collection effort by the Baltimore District would be used to provide the data needed to evaluate the performance of the breakwater in attenuating waves. This effort includes one-directional wave gage and two nondirectional wave gages deployed in a 3- to 4-month wave intensive season. The locations of the wave gages were recently determined based on a Baltimore District wave modeling study.

The directional wave gage is located outside the breakwater, and the nondirectional wave gages are located just inside the breakwater opening and at the county boat dock where most vessels are moored. The wave observations would be correlated with the analytical and numerical model results to determine the accuracy of the models as a design tool for such projects.

Tide data

Continuous recording tide gages would be located both inside and outside of Tedious Creek. The outside gage would be used to define a source tide that would be used to drive revalidation simulations of the tidal circulation models. The inside gage would be used as a validation gage. One-week data collection exercises are planned in each of the first 3 years. Data would be collected at 10-min intervals during spring range tide conditions.

Current data

Current velocity data would be collected using overboard acoustic doppler current profiler (ADCP) meters, and other methods as necessary around the breakwaters, to provide a data set with which the tidal circulation models can be validated. One-week data collection exercises are planned for the first 3 years. The data will be collected at half-hourly intervals during spring range tide conditions.

Sediment data

Bottom grab samples of sediment would be collected throughout the Tedious Creek area to determine sedimentation patterns. The locations would be biased to areas predicted by the circulation models to be areas of anticipated scour or deposition. Coarser materials are expected to occur in areas of higher velocities such as gaps in the breakwaters. Finer materials are expected in lower velocity areas near the breakwaters sheltered from

tidal currents. Grain-size analyses would be performed and ultimately used in the numerical models to determine their ability to predict such processes.

During the course of the study, project performance will be evaluated to determine lessons learned that will apply to other navigation projects, both regionally and nationwide. Technical notes will be published periodically to expeditiously disseminate information to the field, and a final report will be prepared upon completion of the study.

2 Previous Study

Purpose

Following completion of the Tedious Creek breakwater project in 1997, Dorchester County and local watermen complained that the project did not perform as anticipated, and was not providing the authorized level of protection. To respond to these local concerns, the Baltimore District contracted with Offshore and Coastal Technologies Incorporated—East Coast to conduct a wave study to ascertain whether the actual performance of the as-built breakwater was satisfactory as compared to the predicted performance of the authorized project.

The study by Offshore and Coastal Technologies consisted of a field data collection component to measure the characteristics of waves impacting the project site, and a wave-modeling component to predict the effectiveness of the breakwater in reducing waves heights that could be expected to occur over the 50-year project life.

Three wave gages were deployed at Tedious Creek for a 3-month period from June to August 2001. One directional wave gage was deployed outside of the breakwater to measure incident wave height, period, and direction. Two nondirectional wave gages were deployed within the Tedious Creek harbor area to measure transformed wave heights and periods in the vicinity of the docking facilities and mooring areas. The data collected from these gages was intended to define wave conditions impacting the project site under the existing conditions (breakwater as-built with 122-m (400-ft) gap) and for use in validating the wave model. Unfortunately, the summer data collection effort did not result in any significant (measurable) wave events over the period of the gage deployment. However, the field data, observations, and numerical sensitivity studies indicated that conditions preventing operations at the county docking facility often resulted from northwesterly waves generated locally in the creek, rather than by waves entering the breakwater gap.

Offshore and Coastal Technologies study[1]

The Baltimore District requested that Offshore & Coastal Technologies develop, calibrate, and verify a wave model for that area. The model would be used to perform wave transformation simulations in the vicinity of the harbor. The study concentrated on conditions occurring at the county boat dock where fishing operations are affected by waves reported to be entering the harbor through the jetty gap.

Specifically, the wave model was to be applied to evaluate wave conditions within the harbor and at the county boat dock facilities under three scenarios:

1. Preproject conditions (no breakwater).
2. Existing conditions (as-built breakwater with 122-m (400-ft) gap).
3. Authorized project (breakwater with 91-m (300-ft) gap).

Data were collected at three sites for validation of the wave model (Figure 7). Because the summer data collection effort (June to August 2001) did not experience a significant wave event, numerical models were employed in a sensitivity study to (a) evaluate the effectiveness of the present situation, (b) evaluate narrowing of the present gap to the authorized project width, and (c) evaluate a theoretical narrowing to a 61-m (200-ft) width.

SMS setup for Tedious Creek

The Corps of Engineers' shallow-water directional spectral wave model was used for this project, as configured for the SMS graphical user interface. The model (STeady state spectral WAVE (STWAVE)) is generally available without diffraction terms; however, a preliminary version was made available that includes diffraction (STB3), so both were applied to the Tedious Creek study. The bathymetry and associated finite difference grid that was adopted for both the diffraction (STB3) and no-diffraction (STWAVE) model versions were the same. The grid spacing was 15.24 m (50 ft).

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[1] This section is extracted essentially verbatim from Offshore and Coastal Technologies Incorporated (2001).

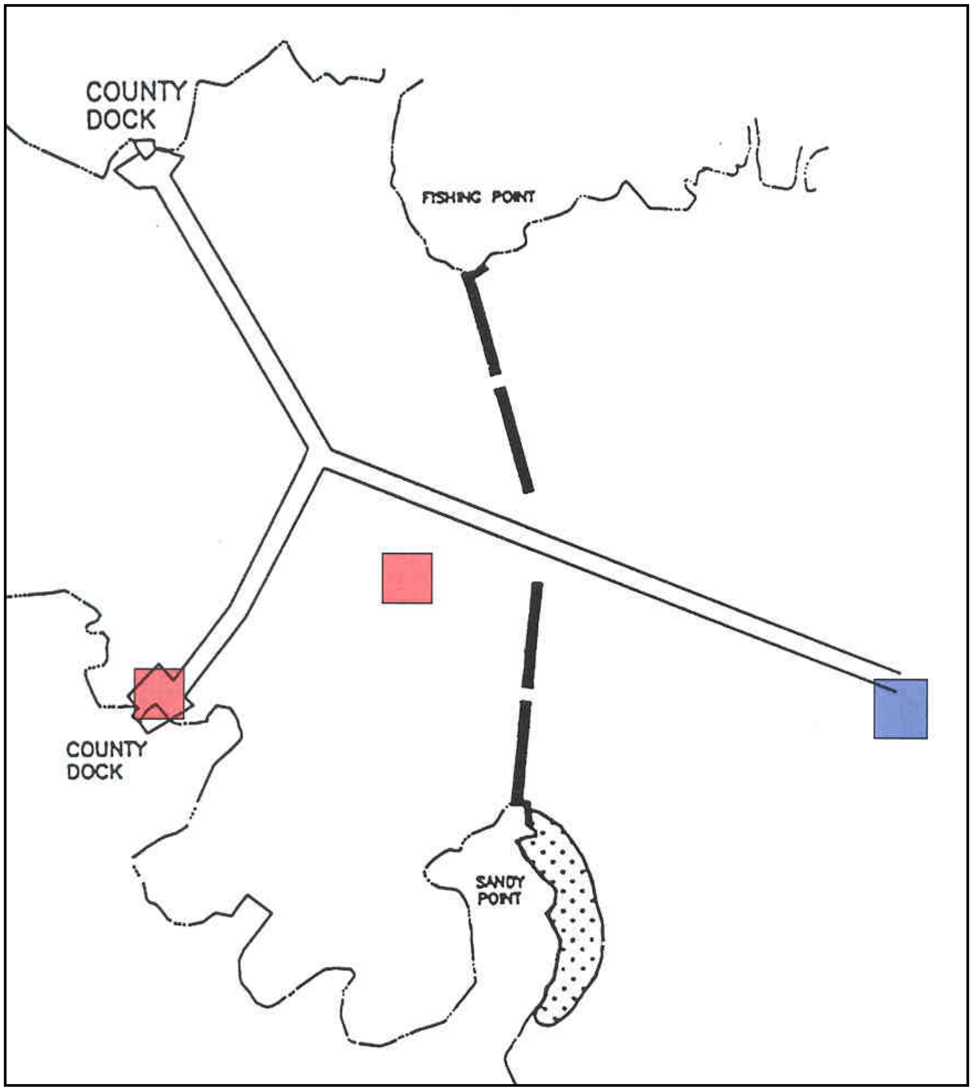


Figure 7. Locations of Offshore & Coastal Technologies wave gages (blue = directional gage placed on channel marker; red = nondirectional gages placed on existing piling).

The breakwater was incorporated into the existing as-built condition simulations, and also into the authorized project simulations, based upon a structure crest survey performed by Offshore & Coastal Technologies. When gridded, the structure was represented in the finite difference grid as two rows with depths of -1.8 m (-6.0 ft) (negative depths in STWAVE are actually above the water level and, in this case, the structure crest is modeled at 1.8 m (6.0 ft) above mean lower low water (mllw)). The structure crest survey indicated that the existing (as-built) gap between the jetties is 122 m (400 ft). Preliminary simulations of wave conditions with the authorized project gap width of 91 m (300 ft) did not cause much of a difference in the waves at the county boat dock. So, to determine some sensitivity to gap width, a gap width of 61 m (200 ft) also was modeled,

along with the case with no breakwater. STB3 (the diffraction version of the code) requires an extra line of information in the input “dep” file containing either zeroes or ones for each row of the model, depending on whether the full series solution for diffraction is implemented or a truncated series is used. The rows immediately behind the breakwater (rows 61, 62, 63, and 64) were modeled with ones, and all other rows were modeled with zeroes.

Since both STWAVE and STB3 models are spectral wave models, the wave frequency spectrum used for all runs was a Joint North Sea Wave Observation Project (JONSWAP) spectrum with a peak enhancement factor of 3.3, represented by the following 30 frequencies:

0.080 0.090 0.100 0.110 0.120 0.130 0.140 0.150 0.160 0.170
0.180 0.190 0.200 0.210 0.220 0.230 0.240 0.250 0.260 0.270
0.280 0.290 0.300 0.310 0.320 0.330 0.340 0.350 0.360 0.370

The directional spectrum was represented by a Mitsuyasu-type wave spreading with a spreading coefficient of 2, or equivalently, cosine-4th spreading. The spreading was symmetric about the principal directions used in the modeling. The directional spectra were represented by 35 five-degree directional components from -85 to +85 deg.

STWAVE and STB3 model runs

The Baltimore District requested that normal and storm conditions be investigated. The following tide, surge, and wave conditions are provided in Appendix C of the U.S. Army Engineer District, Buffalo (1995), feasibility report:

- Mean tide range is about 0.7 m (2.4 ft).
- Spring tide range is about 0.9 m (3.0 ft).
- 5-year water level is about 1.1 m (3.7 ft) above mllw.
- 100-year water level is about 1.8 m (6.0 ft) above mllw.
- 5-year storm wave height from the northeast to southeast ranges from 0.5 to 0.7 m (1.6 to 2.3 ft).
- 50-year storm wave height from the northeast to southeast ranges from 0.6 to 1.5 m (2.1 to 4.9 ft).

These statistics indicate that in order to bring any wave energy of significance into the harbor, some conservatism is required in specifying

modeling conditions because of the very shallow depths approaching the jetties and within the harbor. During minor to moderate winter storms, waves entering through the gap propagate to the county boat dock and inhibit operations. So most computer runs in this study concentrated on the “moderate” storm waves and worst-case directions.

To create a reasonable model run matrix, Table 1 summarizes the wave height, direction, and water level cases run with both the diffraction and no-diffraction versions of STWAVE. At the extreme water level of +1.8 m (+6 ft), the water level was just up to the crest of the jetties. This is an extremely rare event at Tedious Creek and is used only to demonstrate the sensitivity of wave energy passing through the gap as a function of gap width. At lower water levels, the model most likely represents the conditions that actually occur (within the accuracy of the model theory).

Table 1. STWAVE input parameters, Tedious Creek, MD.

H_{m0} (ft)	Directions*	Water Level (ft)	Condition
3.28	0, 10, -10, 22.5, -22.5, 30, -30, 45, -45	0, 2.4	Low tide and high tide storm
1.6	45	1.6	Daily wave, mid tide
1.6	45	3.3	Storm at spring tide
6.26	45	6	Extreme storm

* 0 (zero) represents waves from the east with positive directions being from progressively more northerly directions and negative directions being from progressively more southerly directions. Waves from 45 deg are considered among the worst cases for the county boat dock area and are used for all test case combinations.

STWAVE and STB3 results for as-built conditions (gap width = 122 m (400 ft))

STWAVE (no-diffraction) and STB3 (diffraction) models of the as-built conditions (122-m (400-ft) gap width) were first implemented to provide:

- Assessment of the expected performance of the present jetty configuration.
- Basis for validation against field data.
- Basis for comparison of the present configuration against the authorized project gap width (91 m (300 ft)).

The as-built breakwater configuration and STWAVE grid is illustrated in Figure 8. Because the tide is a large percentage of the total depth in the

area (i.e., depths in the gap are about 1.2 m (4 ft) mllw and the tide range is 0.7 m (2.4 ft)), STWAVE runs were made at both high and low tide. The model water depth at the county boat dock is about 0.3 m (1 ft) at mllw and 1.0 m (3.4 ft) at high tide. Table 2 illustrates the list of computer simulations, including the cases where the offshore wave direction was varied along with the tide.

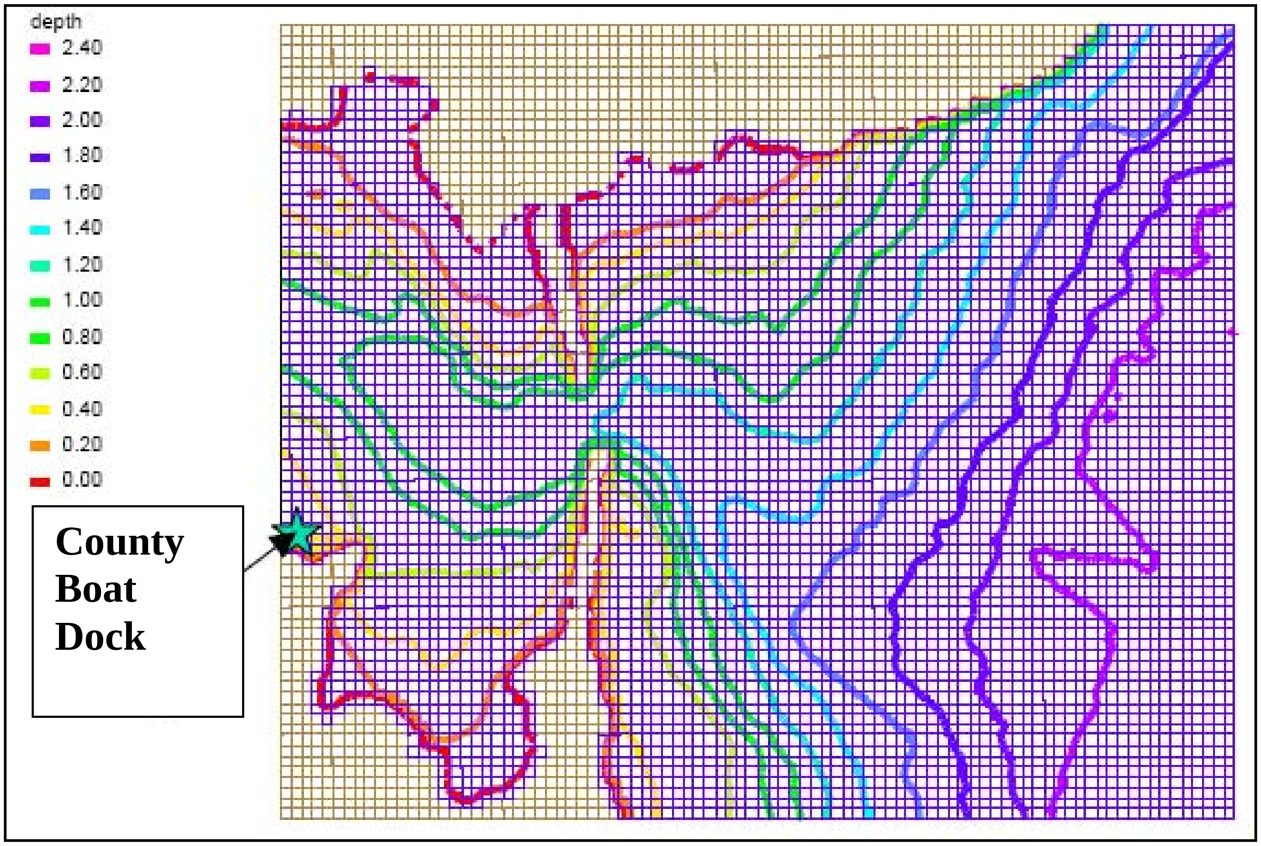


Figure 8. STWAVE and STB3 grid and as-built breakwater configuration of gap width = 122 m (400 ft) (bottom depths in meters (mllw) with no tide).

Table 2. Ratio of wave height at center of 122-m (400-ft) breakwater gap, and at the county boat dock, to the offshore wave height for incident wave height of 1 m (3.3 ft) (varying wave direction and tide).

Model	Wave Direction	Point	Tide 0.0 ft	Tide 2.4 ft
STWAVE	0 (E)	Centerline of gap	0.41	0.69
	10		0.42	0.7
	-10		0.41	0.67
	22.5 (ENE)		0.42	0.7
	-22.5 (ESE)		0.41	0.63
	30		0.42	0.68
	-30		0.4	0.6
	45 (NE)		0.42	0.61
	-45 (SE)		0.39	0.51
	0 (E)	Dock	0.09	0.13
	10		0.09	0.15
	-10		0.08	0.12
	22.5 (ENE)		0.1	0.17
	-22.5 (ESE)		0.08	0.1
	30		0.1	0.18
	-30		0.08	0.09
	45 (NE)		0.11	0.18
	-45 (SE)		0.07	0.07
STB3	0 (E)	Centerline of gap	0.37	0.68
	10		0.37	0.75
	-10		0.37	0.71
	22.5 (ENE)		0.38	0.71
	-22.5 (ESE)		0.37	0.73
	30		0.39	0.64
	-30		0.37	0.7
	45 (NE)		0.4	0.52
	-45 (SE)		0.37	0.6
	0 (E)	Dock	0.1	0.19
	10		0.1	0.2
	-10		0.1	0.18
	22.5 (ENE)		0.11	0.18
	-22.5 (ESE)		0.1	0.16
	30		0.12	0.16
	-30		0.1	0.15
	45 (NE)		0.12	0.13
	-45 (SE)		0.09	0.13

Table 2 shows that for existing conditions (122-m (400-ft) gap width) and a 1-m (3.3-ft) incident wave height (a 1-m "unit" wave height), both models indicate:

- 30 to 40 percent of the incident wave height enters the harbor through the gap at low tide.
- 50 to 70 percent of the incident wave height enters the harbor through the gap at high tide.
- As much as 12 percent of the incident wave height reaches the county boat dock at low tide.
- As much as 18 percent of the incident wave height reaches the county boat dock at high tide.
- A wave arriving from between 0 and 30 deg north of east results in the highest waves at the county boat dock.

STWAVE results for preconstruction (no breakwater) conditions

STWAVE was implemented for the no-breakwater (preconstruction condition) to provide an estimate of the wave heights experienced by the area prior to construction of the breakwater. The diffraction model STB3 was not applied here because no diffractive structures were present.

Preconstruction wave patterns are illustrated in Figure 9 for waves from the northeast. Again, because the tide is a large percentage of the total depth in the area, STWAVE runs were made at both high and low tides. Table 3 illustrates the list of computer simulations, including the cases where the offshore wave direction was varied along with the tide.

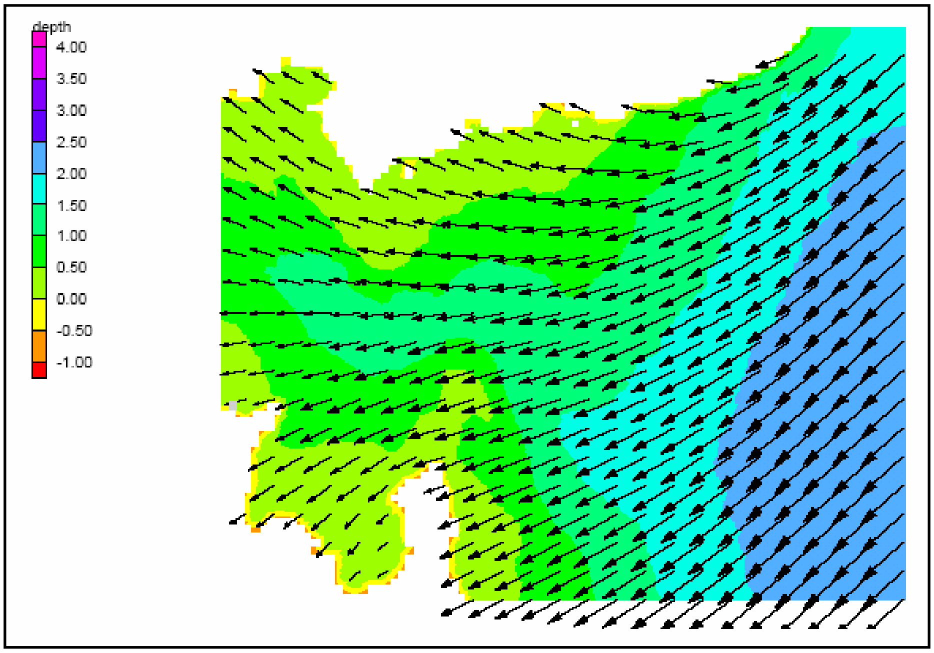


Figure 9. Patterns of waves arriving from the northeast for the no-breakwater condition (bottom depths in meters (mllw) with no tide).

Table 3. Ratio of wave height at center of harbor entrance (no breakwater), and at the county boat dock, to the offshore wave height for incident wave height of 1 m (3.3 ft) (varying wave direction and tide).

Model	Wave Direction	Point	Tide 0.0 ft	Tide 2.4 ft
STWAVE	0 (E)	Centerline of gap	0.44	0.73
	10		0.44	0.74
	-10		0.44	0.71
	22.5 (ENE)		0.44	0.73
	-22.5 (ESE)		0.43	0.68
	30		0.44	0.7
	-30		0.43	0.65
	45 (NE)		0.44	0.63
	-45 (SE)		0.42	0.58
	0 (E)	Dock	0.15	0.33
	10		0.15	0.33
	-10		0.15	0.32
	22.5 (ENE)		0.15	0.32
	-22.5 (ESE)		0.15	0.31
	30		0.15	0.31
	-30		0.15	0.3
	45 (NE)		0.15	0.28
	-45 (SE)		0.15	0.27

Table 3 shows that for existing conditions and a 1-m (3.3-ft) incident wave height (a 1-m "unit" wave height), the model indicates:

- 43 to 44 percent of the incident wave height enters the harbor at low tide.
- 58 to 74 percent of the incident wave height enters the harbor through the gap at high tide.
- 15 percent of the incident wave height reaches the county boat dock at low tide.
- 27 to 33 percent of the incident wave height reaches the county boat dock at high tide.

STWAVE and STB3 results for authorized project (gap width = 91 m (300 ft)

STWAVE (no-diffraction) and STB3 (diffraction) models of the authorized project condition (91-m (300-ft) gap width) were implemented to provide an assessment of the potential reduction in wave height at the county boat dock if the present gap was narrowed by an additional 30.5 m (100 ft) from the existing 122 m (400 ft) to the designed authorized project gap of 91 m (300 ft).

Wave patterns for waves from the northeast with the authorized project breakwater configuration are illustrated in Figure 10. For computer runs, again both models were run at mllw and with a high tide of 0.7 m (2.4 ft). Table 4 illustrates the list of computer simulations, including the cases where the offshore wave direction was varied along with the tide.

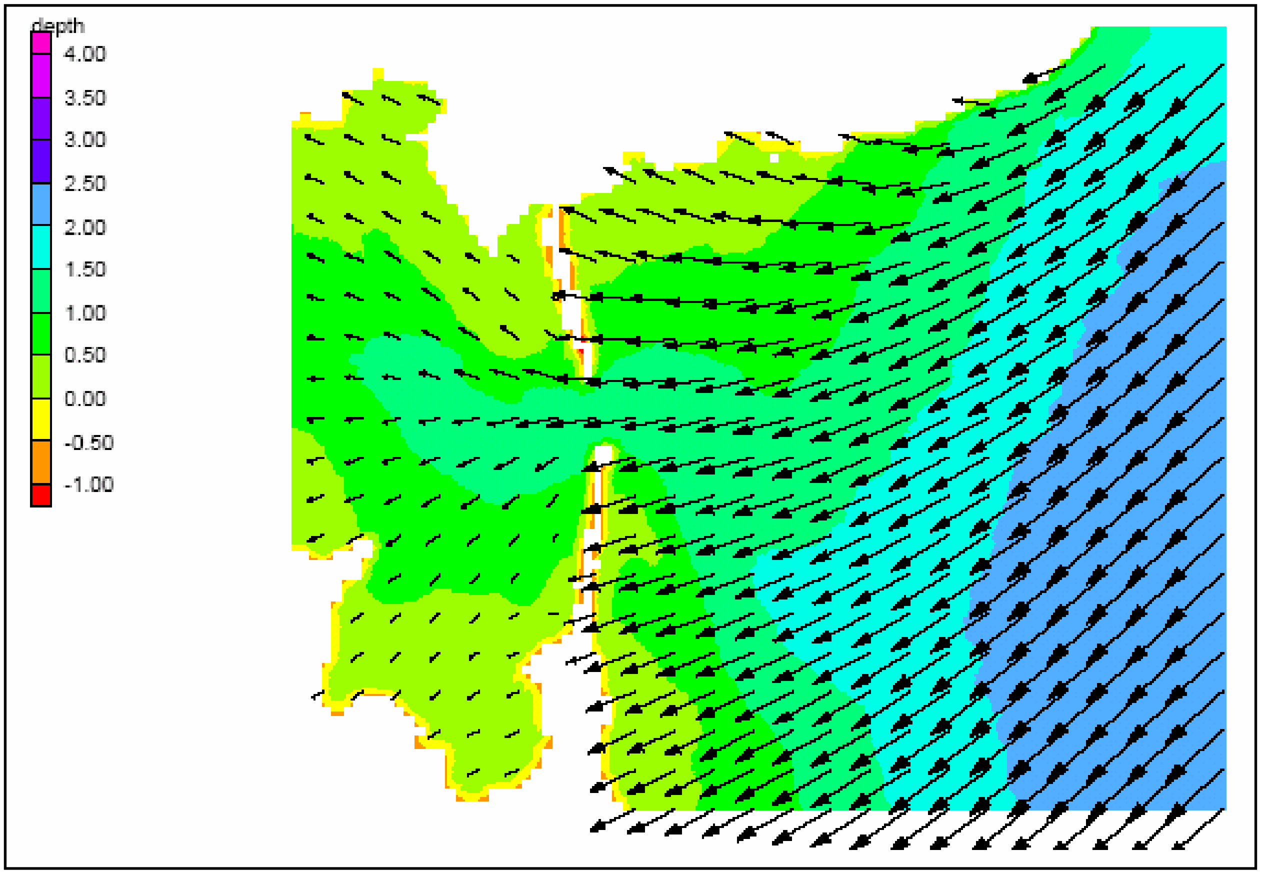


Figure 10. Patterns of waves arriving from the northeast for the authorized project condition of gap width = 91 m (300 ft) (bottom depths in meters (mllw) with no tide).

Table 4. Ratio of wave height at center of 91-m (300-ft) breakwater gap, and at the county boat dock, to the offshore wave height for incident wave height of 1 m (3.3 ft) (varying wave direction and tide).

Model	Wave Direction	Point	Tide 0.0 ft	Tide 2.4 ft
STWAVE	0 (E)	Centerline of gap	0.41	0.69
	10		0.42	0.7
	-10		0.41	0.67
	22.5 (ENE)		0.42	0.7
	-22.5 (ESE)		0.41	0.63
	30		0.42	0.68
	-30		0.4	0.6
	45 (NE)		0.42	0.61
	-45 (SE)		0.39	0.51
	0 (E)	Dock	0.09	0.13
	10		0.09	0.15
	-10		0.08	0.12
	22.5 (ENE)		0.1	0.17
	-22.5 (ESE)		0.08	0.1
	30		0.1	0.18
	-30		0.08	0.09
	45 (NE)		0.11	0.18
	-45 (SE)		0.07	0.07
STB3	0 (E)	Centerline of gap	0.37	0.68
	10		0.37	0.75
	-10		0.37	0.71
	22.5 (ENE)		0.38	0.71
	-22.5 (ESE)		0.37	0.73
	30		0.39	0.64
	-30		0.37	0.7
	45 (NE)		0.4	0.52
	-45 (SE)		0.37	0.6
	0 (E)	Dock	0.1	0.18
	10		0.1	0.19
	-10		0.1	0.17
	22.5 (ENE)		0.11	0.16
	-22.5 (ESE)		0.09	0.16
	30		0.12	0.15
	-30		0.09	0.14
	45 (NE)		0.12	0.13
	-45 (SE)		0.09	0.12

Table 4 shows that for authorized project conditions (91-m (300-ft) gap width) and a 1-m (3.3-ft) incident wave height (a 1-m “unit” wave height), both models indicate:

- 37 to 42 percent of the incident wave height enters the harbor through the gap at low tide.
- 51 to 75 percent of the incident wave height enters the harbor through the gap at high tide.
- As much as 12 percent of the incident wave height reaches the county boat dock at low tide.
- As much as 19 percent of the incident wave height reaches the county boat dock at high tide.
- Waves arriving from between 10 and 45 deg north of east result in the highest waves at the county boat dock.

STWAVE and STB3 results for smaller “daily” wave heights

Because all the 1-m (3.3-ft) wave height model results indicate that the highest waves at the county boat dock result from waves arriving from the northeast quadrant (waves from east to the northeast), additional simulations were run with smaller wave heights for northeasterly waves. This phase of the analysis was performed to assess the sensitivity of the model results to wave breaking processes included in the models. The breakwater gap was also reduced an additional 30.5 m (100 ft) so that the gap would be 61 m (200 ft) to investigate if further narrowing of the gap would provide significant additional benefits. Table 5 shows the results for different gap widths with a 0.5-m (1.6-ft) wave height and a water-level increase of 0.5 m (1.6 ft).

Table 5. Ratio of wave height at the center of the breakwater gap, and at the county boat dock, to the offshore wave height for incident wave height of 0.5 m (1.6 ft) from the northeast (tide at 0.5 m (1.6 ft).

Model	Breakwater Width	Point	H_{mo} Ratio
STWAVE	300 ft	Centerline of gap	0.6
		Dock	0.18
STB3	300 ft	Centerline of gap	0.6
		Dock	0.2
STWAVE	400 ft	Centerline of gap	0.6
		Dock	0.18
STB3	400 ft	Centerline of gap	0.6
		Dock	0.2
STWAVE	200 ft	Centerline of gap	0.56
		Dock	0.16
STB3	200 ft	Centerline of gap	0.6
		Dock	0.14
STWAVE	Open	Centerline of gap	0.56
		Dock	0.18

Table 5 shows that for smaller "daily" wave conditions and a tide that is slightly above mid-tide:

- As-built breakwater (gap width = 122 m (400 ft) results in about 18 to 20 percent of the offshore wave height at the county boat dock.
- There is little difference between a no-breakwater condition and having breakwaters with a gap of 91 to 122 m (300 to 400 ft) because of wave breaking and strong refraction toward the north and east within the harbor.
- There is very little difference in the effect of narrowing the breakwater gap from 122 m (400 ft) to 91 m (300 ft).
- A 61-m (200-ft) gap reduces the wave height to 14 to 16 percent of the incident wave height, or about 25 percent less than the waves occurring when the jetties are not present or when the gap is 91 to 122 m (300 to 400 ft).

STWAVE and STB3 results for larger "extreme storm" waves and storm surges

The last series of numerical model runs was made for 2-m (6.6-ft) offshore waves coming from the northeast. A water-level increase of 1.8 m (6.0 ft)

was included. This series of runs is modeled an extreme event, with an estimated 100-year return period for waves and for water levels. This is an extremely rare event but was calculated to provide an ‘upper bound’ on the performance of the breakwater in providing wave protection. These results are summarized in Table 6.

Table 6. Ratio of wave height at the center of the breakwater gap, and at the county boat dock, to the offshore wave height for incident wave height of 2 m (6.6 ft) from the northeast (tide at 1.8 m (6.0 ft).

Model	Breakwater Width	Point	H_{mo} Ratio
STWAVE	300 ft	Centerline of gap	0.63
		Dock	0.26
STB3	300 ft	Centerline of gap	0.61
		Dock	0.19
STWAVE	400 ft	Centerline of gap	0.63
		Dock	0.26
STB3	400 ft	Centerline of gap	0.61
		Dock	0.19
STWAVE	200 ft	Centerline of gap	0.59
		Dock	0.23
STB3	200 ft	Centerline of gap	0.61
		Dock	0.15
STWAVE	Open	Centerline of gap	0.64
		Dock	0.72

Table 6 shows that for “extreme storm” wave conditions and a 100-year storm tide:

- As-built breakwater (gap width = 122 m (400 ft)) results in about 19 to 26 percent of the offshore wave height at the county boat dock as compared to 72 percent of the storm wave height reaching the county boat dock with no breakwater in place for northeasterly waves.
- There is very little difference in the effect of narrowing the breakwater gap to 91 m (300 ft).
- Breakwaters with gaps of 61 to 122 m (200 to 400 ft) provide a reduction in wave height to about 15 to 23 percent, respectively, as compared to about 72 percent at the county boat dock with no breakwater in place for northeasterly waves.

- A 61-m (200-ft) gap reduces the wave height to 15 to 23 percent of the incident wave height, or about a 10- to 20-percent further reduction below the 91- to 122-m (300- to 400-ft) gaps.

Summary and conclusions[1]

Numerical wave models STWAVE (no-diffraction) and STB3 (diffraction) were configured to perform wave transformation simulations in the vicinity of Tedious Creek Harbor, MD. The Baltimore District requested that Offshore and Coastal Technologies Incorporated—East Coast apply the models to evaluate wave conditions within the harbor and at the county boat dock facilities under three scenarios:

- Preproject conditions (no breakwater).
- Existing as-built conditions (122-m (400-ft) breakwater gap opening).
- Authorized project (91-m (300-ft) gap opening).

Wave height transformations were performed with varying wave heights, tides, storm surge levels, and incident wave angles. The simulations concentrated on conditions occurring at the county boat dock where fishing operations are affected by waves reported to be entering the harbor through the jetty gap. The worst wave conditions at the boat dock appear to result from northeasterly offshore waves.

During “extreme storm” wave events (100-year wave and water levels), the existing as-built breakwater with a 122-m (400-ft) gap reduces wave heights at the county boat dock by as much as 70 percent, as compared to the no-breakwater project conditions, from 2 m (6.4 ft) offshore to 0.5 m (1.7 ft) at the county boat dock. Narrowing the as-built breakwater gap width to the authorized project breakwater gap of 91 m (300 ft) resulted in very little difference in the storm wave heights at the county boat dock.

During moderate wave conditions (0.9- to 1.2-m (3- to 4-ft) offshore wave heights), the as-built breakwater reduces wave heights at the county boat dock by as much as 50 percent at high tide and 30 percent at low tide, as compared to the no-breakwater conditions, to a wave height of 0.1 m (0.3 ft) at low tide and 0.2 m (0.6 ft) at high tide. The authorized project

[1] This section was written by Karen M. Nook (2002), Civil Engineer, Engineering Division, U.S. Army Engineer District, Baltimore, MD.

with a gap width of 91 m (300 ft) did not result in any difference in the wave heights at the county boat dock for either low or high tides.

During typical “daily” conditions (0.3- to 0.6-m (1- to 2-ft) offshore wave heights), neither the as-built nor authorized projects result in any reduction in wave heights at the county boat dock at a mid-tide level of 0.5 m (1.6 ft). Wave heights at the county boat dock, however, are transformed by the natural bathymetry of the creek, to a tolerable level of less than 0.2 m (0.5 ft) for no-breakwater conditions, and for all breakwater gap widths. A summary of the wave modeling results is provided in Table 7.

In addition to the scenarios requested by the Baltimore District, Offshore and Coastal Technologies also modeled a hypothetical 61-m (200-ft) gap width. This scenario would require modification of the authorized project to extend the breakwaters an additional 61 m (200 ft) beyond the as-built project or 30.5 m (100 ft) beyond the authorized project. The models STWAVE and STB3 demonstrated that narrowing the breakwater gap width to 61 m (200 ft) would result in an insignificant difference in the storm wave heights at the county boat dock, and a 10-percent reduction in wave heights during typical daily conditions. However, since the as-built project was shown to reduce normal daily wave heights at the county boat dock to less than 0.2 m (0.5 ft), which is considered a tolerable level for vessel unloading and mooring, an additional 10-percent reduction would be considered insignificant, and modification of the project would not be justified.

More significantly, field data and observations made during project location site visits indicate that wave conditions preventing satisfactory operations at the county boat dock facility often result from northwesterly waves generated locally on Tedious Creek, rather than by waves propagating through the breakwater gap from a northeasterly direction.

Table 7. Tedious creek monitoring study by offshore and coastal technologies (2001); summary of wave model results.

Daily Wave Conditions

		Pre-Project No Gap		As-Built 122 m (400 ft) Gap		Authorized Project 91 m (300 ft) Gap	
Wave Location	Water Level	Hmo Ratio	H (ft)	Hmo Ratio	H (ft)	Hmo Ratio	H (ft)
Offshore (Hmo)	+1.64 ft mllw	1	1.64	1	1.64	1	1.64
CL Gap	+1.64 ft mllw	0.56	0.92	0.6	0.98	0.6	0.98
Boat Ramp	+1.64 ft mllw	0.18	0.30	0.18	0.30	0.18	0.30
Percent wave height reduction at county boat ramp					0%		0%

Offshore daily wave height of 0.5 m (1.6 ft) from worst case ENE direction at mid-tide level of +0.5 m (+1.6 ft) mllw
Negligible reduction in wave height at the county boat ramp for both 122 and 91 m (400 and 300 ft) gaps

Moderate Wave Conditions

		Pre-Project No Gap		As-Built 122 m (400 ft) Gap		Authorized Project 91 m (300 ft) Gap	
Wave Location	Water Level	Hmo Ratio	H (ft)	Hmo Ratio	H (ft)	Hmo Ratio	H (ft)
Offshore (Hmo)	0.0 ft mllw	1	3.28	1	3.28	1	3.28
CL Gap	0.0 ft mllw	0.44	1.44	0.42	1.38	0.42	1.38
Boat Ramp	0.0 ft mllw	0.15	0.49	0.1	0.33	0.1	0.33
Percent wave height reduction at county boat ramp					33%		33%
Offshore (Hmo)	+2.4 ft mllw	1	3.28	1	3.28	1	3.28
CL Gap	+2.4 ft mllw	0.73	2.39	0.7	2.30	0.7	2.30
Boat Ramp	+2.4 ft mllw	0.32	1.05	0.17	0.56	0.17	0.56
Percent wave height reduction at county boat ramp					47%		47%

Offshore wave height of 1m (3.3 ft) from worst case ENE direction at normal tide of +0.0 m (+0.0 ft) mllw
200 ft gap was not modeled for moderate wave conditions
About 33 percent reduction in wave height at the county boat ramp at normal tide of + 0.0 m (+0.0 ft) mllw for both 122 and 91 m (400 and 300 ft) gaps
About 47 percent reduction in wave height at the county boat ramp at tide of 0.7 m (+2.4 ft mllw) for both 122 and 91 m (400 and 300 ft) gaps

Storm Wave Conditions

		Pre-Project No Gap		As-Built 122 m (400 ft) Gap		Authorized Project 91 m (300 ft) Gap	
Wave Location	Water Level	Hmo Ratio	H (ft)	Hmo Ratio	H (ft)	Hmo Ratio	H (ft)
Offshore (Hmo)	Tide 6.0 ft	1	6.56	1	6.56	1	6.56
CL Gap	Tide 6.0 ft	0.64	4.20	0.63	4.13	0.63	4.13
Boat Ramp	Tide 6.0 ft	0.72	4.72	0.26	1.71	0.26	1.71
Percent wave height reduction at county boat ramp					64%		64%

Offshore storm wave height of 2 m (6.6 ft) from worst case ENE direction with 1.8 m (+6 ft) mllw storm tide (approximately 100 year wave and water level)
About 64 percent reduction in storm wave height at county boat ramp for both 122 and 91 m (300 and 400 ft) gaps

3 MCNP Field Data Collection and Analysis[1]

High-quality field data measurements are an integral part of the design process for either new projects or existing engineering project modifications. A key to success in numerical hydrodynamic and wave modeling is a field measurement program to obtain as much information as possible about key input parameters. Carefully collected, high-resolution field measurements yield valuable insights to aid in the interpretation of processes active in a project area.

Methods, techniques, and instrument configurations that were employed to obtain field measurements suitable for estimating water-level fluctuations, channel geometry, bed material classification, tidal currents, and marsh-line definition in a numerical study looking at hydrodynamic conditions in the Tedious Creek small boat harbor are described. These techniques follow Pratt et al. (1999). This study evaluated the performance of the breakwater in front of the small boat harbor and looked at sedimentation issues as they relate to shoreline degradation and channel maintenance dredging.

Seven different types of field data collection were conducted over a 2-year period: (a) positioning and datum referencing, (b) aerial photography, (c) tidal data collection, (d) wave data collection, (e) surveys (hydrographic, bank line, and breakwater), (f) bottom sample collection, and (g) current velocities. Dates of the field data collections were August 2001, September 2002, March 2003, May 2003, and August 2003.

Positioning and data referencing

Locating and verifying horizontal and vertical control is an important part of any data collection effort. At this location, the vertical control was suspect because of local subsidence. There are several benchmarks that have anchor points driven to a point of refusal in the area. The National Ocean Service (NOS) Web site, *http://co-ops.nos.noaa.gov/*, provides users the necessary monument descriptions and locations within a project

[1] This chapter was written by Michael W. Tubman, Research Hydraulic Engineer, Flood and Storm Protection Division, Field Data Collection and Analysis Branch, CHL, with pertinent extractions from Pratt (2003).

area. Sometimes these benchmarks are hard to find, so it is advisable to get the descriptions of as many as possible in the area (Figure 11).

U.S. DEPARTMENT OF COMMERCE
National Oceanic and Atmospheric Administration
National Ocean Service

TIDAL BENCHMARKS

Station ID: 8571559 PUBLICATION DATE: 09/26/2000
Name: MCCREADYS CREEK, FISHING BAY, MARYLAND
NOAA Chart: 12261 Latitude: 38° 18.0' N
USGS Quad: WINGATE Longitude: 76° 0.4' W

BENCHMARK STAMPING: 1559 B 1981
DESIGNATION: 857 1559 B

MONUMENTATION: Benchmark disk VM#: 3066
AGENCY: National Ocean Service (NOS) PID:
SETTING CLASSIFICATION: Stainless steel rod

The benchmark is a disk located near the picnic area, 65.5 m (215 ft) NW of the north end of the west bulkhead for the boat ramp, 27.52 m (90.3 ft) west of the center line of the road leading to McCreadys Point, 4.54 m (14.9 ft) WNW of the NE corner of the concrete slab for the shaded picnic area, and 4.15 m (13.6 ft) ENE of the NW corner of the concrete slab for the picnic area. The benchmark is crimped to the top of a stainless steel rod driven 17.1 m (56 ft) to refusal.

BENCHMARK STAMPING: 1559 C 1981
DESIGNATION: 857 1559 C

MONUMENTATION: Benchmark disk VM#: 3067
AGENCY: National Ocean Service (NOS) PID:
SETTING CLASSIFICATION: Stainless steel rod

The benchmark is a disk located along the entrance road to McCreadys Point, 58.00 m (190.3 ft) NNW of the north end of the asphalt parking area, which is parallel to the north end of the shaded picnic area, and 4.33 m (14.2 ft) east of the center line of the road. The benchmark is crimped to the top of a stainless steel rod driven 19.2 m (63 ft) to refusal.

BENCHMARK STAMPING: 1559 D 1981
DESIGNATION: 857 1559 D

MONUMENTATION: Benchmark disk VM#: 3068
AGENCY: National Ocean Service (NOS) PID:
SETTING CLASSIFICATION: Stainless steel rod

The benchmark is a disk located along the road leading to McCreadys Point, 63.4 m (208 ft) NNW of Benchmark 1559 C 1981 and 3.84 m (12.6 ft) east of the center line of the road. The benchmark is crimped to the top of a stainless steel rod driven 16.2 m (53 ft) to refusal.

Figure 11. Descriptions of NOS tidal benchmarks, vicinity of Tedious Creek, MD (after Pratt 2003).

The construction benchmarks near the jetty structure were surveyed to these NOS tidal benchmarks using a Real Time Kinematic – Global Positioning System (RTK-GPS) system. This was done to check the validity of the construction benchmarks and to determine the appropriate offsets for locating the GPS base station onsite while conducting all survey operations. These offsets would also provide a check in future surveys in the area to monitor subsidence at the site since the first order points are driven to a point of refusal.

Aerial photography

Targets are first priority in obtaining precision aerial photography because a multitude of factors are involved. Preflight logistics of setting viewable targets must be completed so that, when the weather is appropriate and at the right stage of the tide, the aerial crew can collect photographic data. The aerial targets were painted on road intersections around the jobsite using white paint. To rectify an image, the user must have points as spatially diverse as possible. Ideally, a point in each corner of the image gives excellent results. The roads in the area were very light colored, so the targets were outlined with black paint. This procedure better defined the targets for processing. The targets were made from two lines that crossed at right angles; the lines were 0.61 m (2 ft) wide and 3.05 m (10 ft) long. The black paint was used to make a 10-cm- (4-in.-) wide border completely around the perimeter of the target (Figure 12). Target center positions were surveyed using RTK-GPS after all locations were painted and complete.

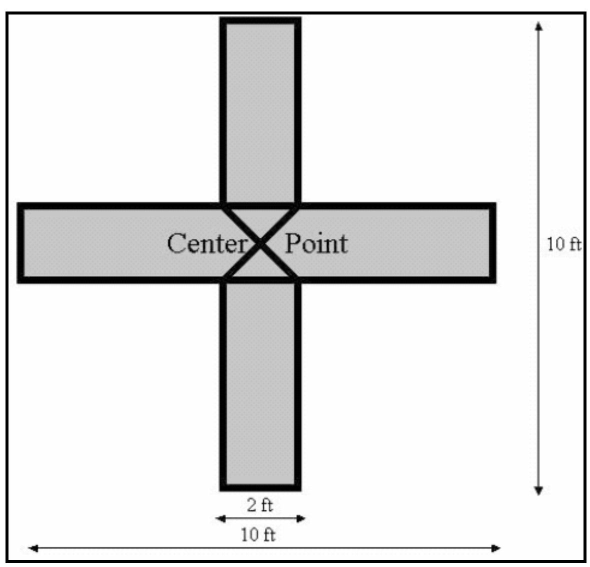


Figure 12. Field targets used to rectify aerial photography (after Pratt 2003).

These data were acquired in the fall of 2001 to define the wetted perimeter at an instant in time, and to monitor the shoreline degradation by comparing surveys from future years. If the shoreline position changed significantly between subsequent surveys, then the erosion or deposition rates could be estimated. The degree of accuracy of the data usually drives the cost of the

collection and dictates the methodology employed. Since meter-level accuracy was sufficient for the numerical modeling requirements at this project location, standard aerial photographic techniques were employed. Figure 13 shows an example of the rectified final product delivered by the aerial photographer.

Figure 13. Aerial mosaic of Tedious Creek, MD, region over-flight data (after Pratt 2003).

Tidal data collection

Water-level data were acquired by pressure tide gages deployed at three locations during the field studies in August 2001 and August 2003. Figure 14 provides the tide gage locations with color contours of bathymetry (ft). Custom aluminum mounting brackets were fixed to stationary piles to deploy the gages. Once deployed, the elevation to the top of each gage was established using the RTK-GPS. These tide gages were programmed to collect changes in water elevation every 15 min. During processing, the established elevation and the change in the water surface were used to process the hydrographic data. These data can be used to produce an accurate graph of the tidal cycle (Figure 15). These measurements provide short-term records of a few days for comparing numerical model results, and for correcting bathymetric survey data. In addition to these short-term data, water surface elevations were recorded for several weeks at a time (Figure 16).

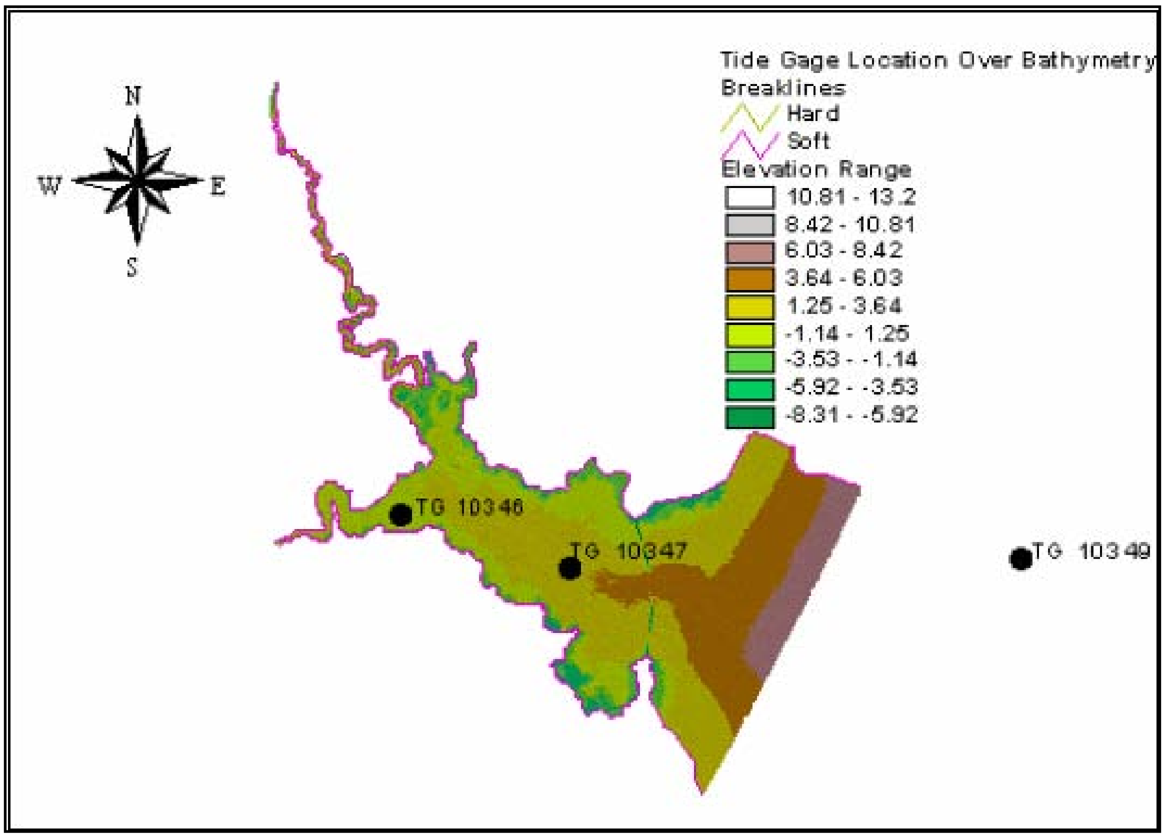


Figure 14. Tide gage locations, Tedious Creek, MD (after Pratt 2003).

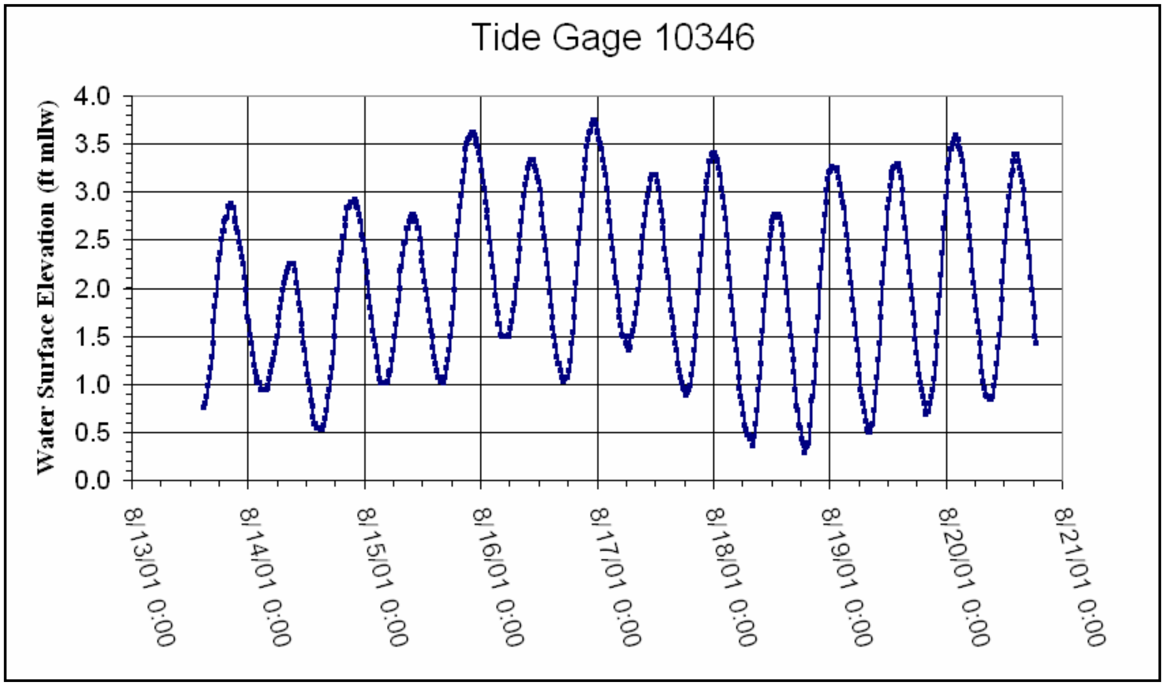


Figure 15. Relatively short-term tidal fluctuations at tide gage 10346, Tedious Creek, MD, 13-21 August 2001.

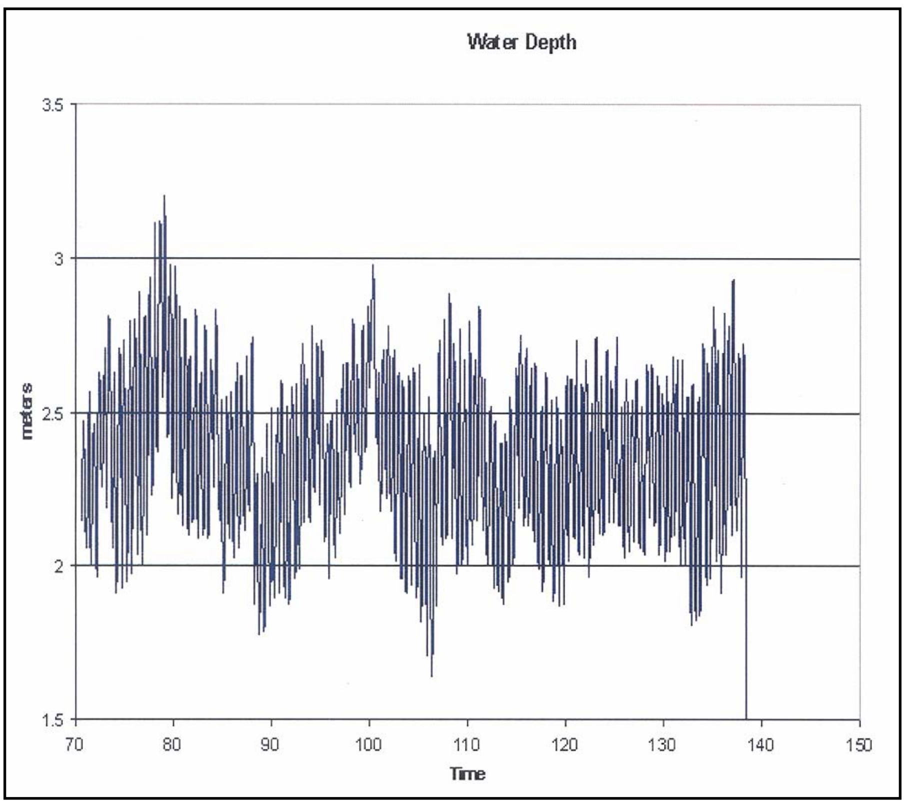


Figure 16. Relatively long-term tidal fluctuations at tide gage 10346, Tedious Creek, MD, 12 March - 20 May 2003 (time in calendar days after 1 January 2003).

Wave data collection

Three wave gages were deployed on 12 March 2003 and recovered on 20 May 2003. Their locations are shown in Figure 17. The two gages inside the jetties were nondirectional gages attached to pilings that recorded 2048 measurements of pressure at a 2-Hz rate every 2 hr. The pressure measurements were converted to surface elevations when the data were processed. The wave gage outside the jetty was a bottom-mounted Sontek Acoustic Doppler Velocimeter with a pressure sensor. Directional wave data are obtained from this instrument from nearly collocated measurements of wave-induced velocity and pressure. The wave heights come from the pressure measurements, after converting them to surface elevations during data processing. The significant wave heights and wave directions plotted by Julian day at the peaks of the wave power spectrums are shown in Figures 18 and 19.

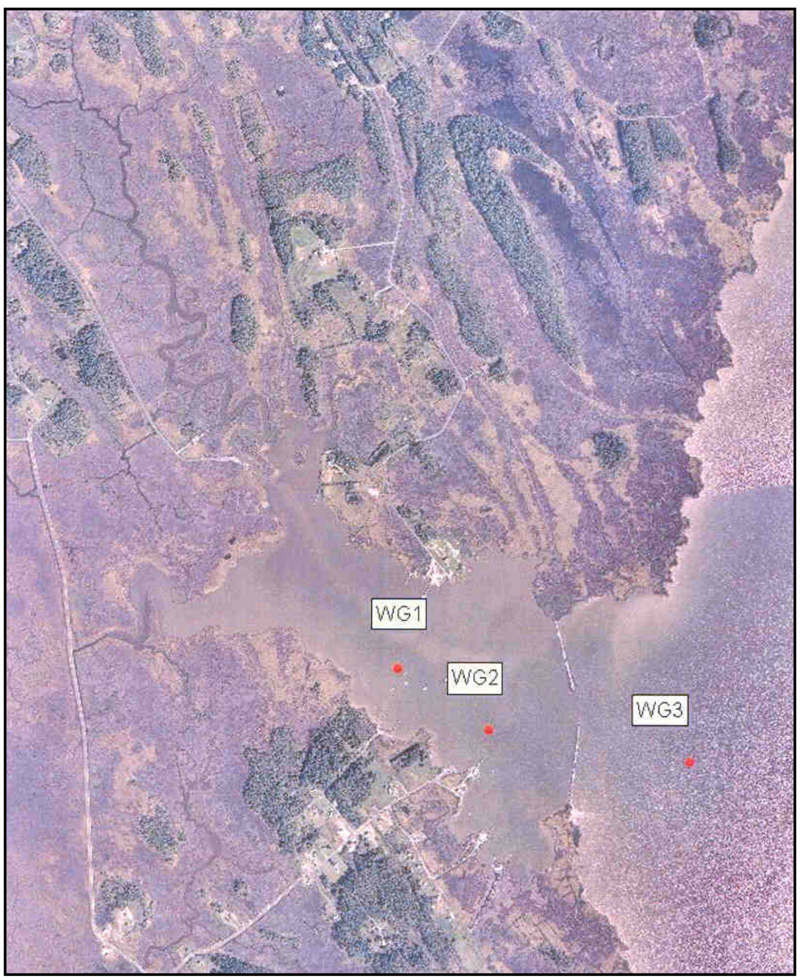


Figure 17. Wave gage locations, Tedious Creek, MD, 12 March – 20 May 2003.

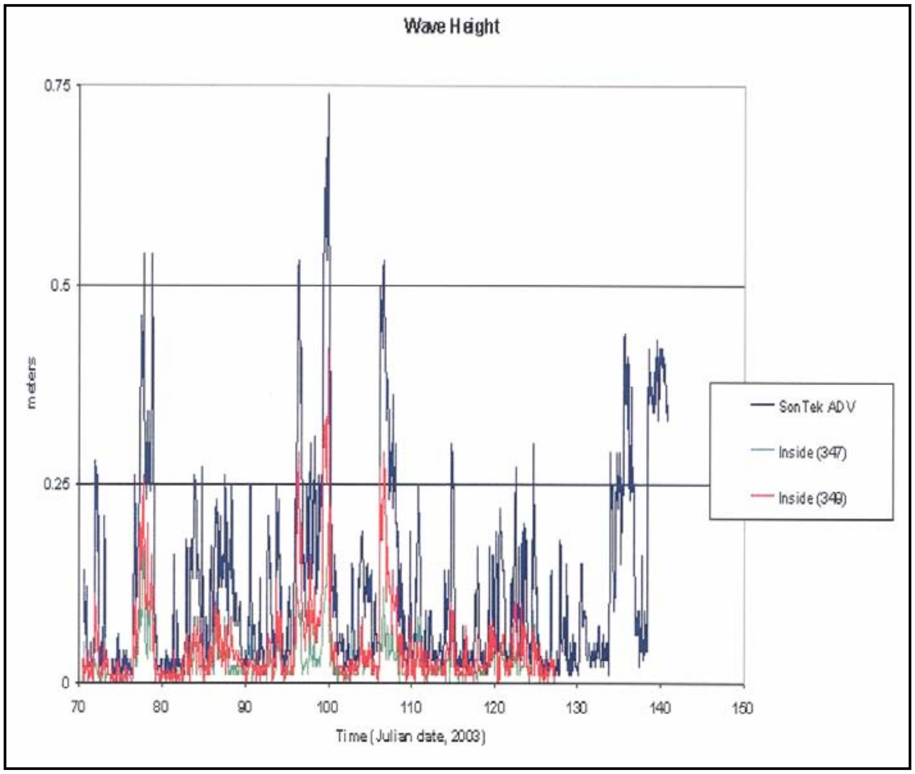


Figure 18. Significant wave heights at peaks of wave power spectrum, wave gage WG3, Tedious Creek, MD, 12 March – 20 May 2003 (time in calendar days after 1 January 2003) (from Figure 17, SonTec ADV = WG3, Inside 347 = WG2, Inside 348 = WG1).

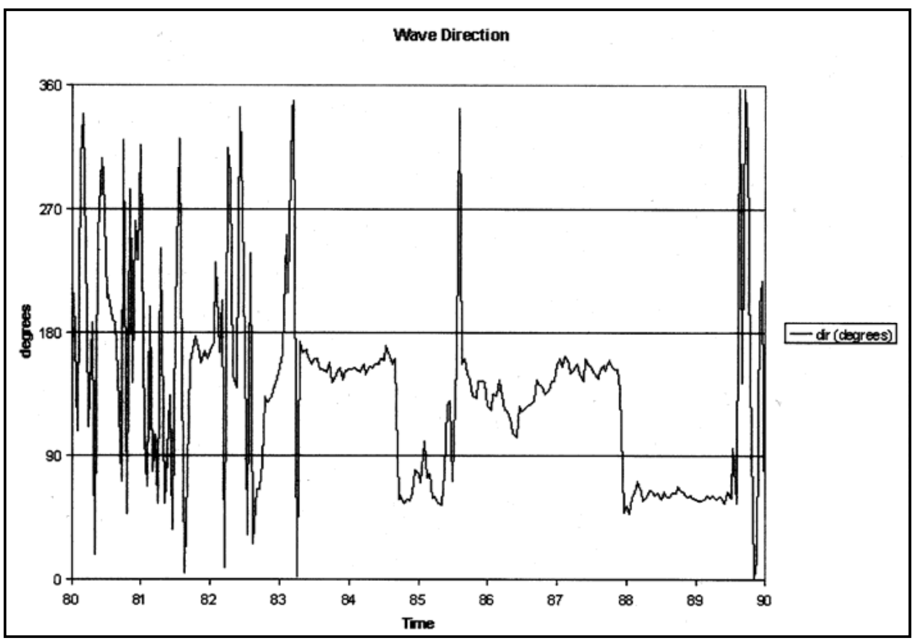


Figure 19. Wave approach direction azimuths at peak of wave power spectrum, wave gage WG3, Tedious Creek, MD, 22 March – 4 April 2003 (time in calendar days after 1 January 2003).

Surveys

Hydrographic surveys

A flat-bottom, 5.5-m (18-ft) survey vessel was used to collect bathymetry data. This vessel was chosen for its shallow draft and ease of use for all aspects of this investigation. The boat was equipped with GPS (for positioning), an Odom Hydrotrac Fathometer (for depth), and a laptop computer using HyPACK (hydrographic data collection software). The hydro-survey crew collected data along lines that had predetermined positions. The lines were drawn digitally, in the survey program, onto maps provided by the Baltimore District. The hydro-survey crew piloted the boat along each line collecting both position and water depth data simultaneously. While collecting hydrographic data, the crew was able to monitor its position along the line. Figure 20 shows the data collection lines for the bathymetry data. The line density around the jetties and channel areas was increased to better define the bathymetry. Figure 21 is a contour plot of the bathymetry data collected along the survey lines. This contour display was produced inside the Hydraulic Processes Analysis System (HyPAS) (Pratt and Cook 1999) after the data were imported.

A second bathymetric survey was conducted in August 2003 to ascertain any changes that may have occurred in this region since the original bathymetric survey that had been conducted in August 2001. There were no detectable significant differences between the results of the two surveys.

Bank line surveys

The survey vessel was used to transport personnel around the perimeter of the Tedious Creek area. A member of the field crew stood on the front of the boat with the RTK-GPS unit and, as the boat pulled into the bank, would collect a data point at the water line. These bank line data were collected approximately every 15.2 m (50 ft) around the perimeter of the bay. Figure 22 shows the point density around the perimeter of the bay and along the rock jetties. Bank line surveys were conducted in August 2001 and August 2003. There were no statistically significant changes in the bank line elevations between the two surveys.

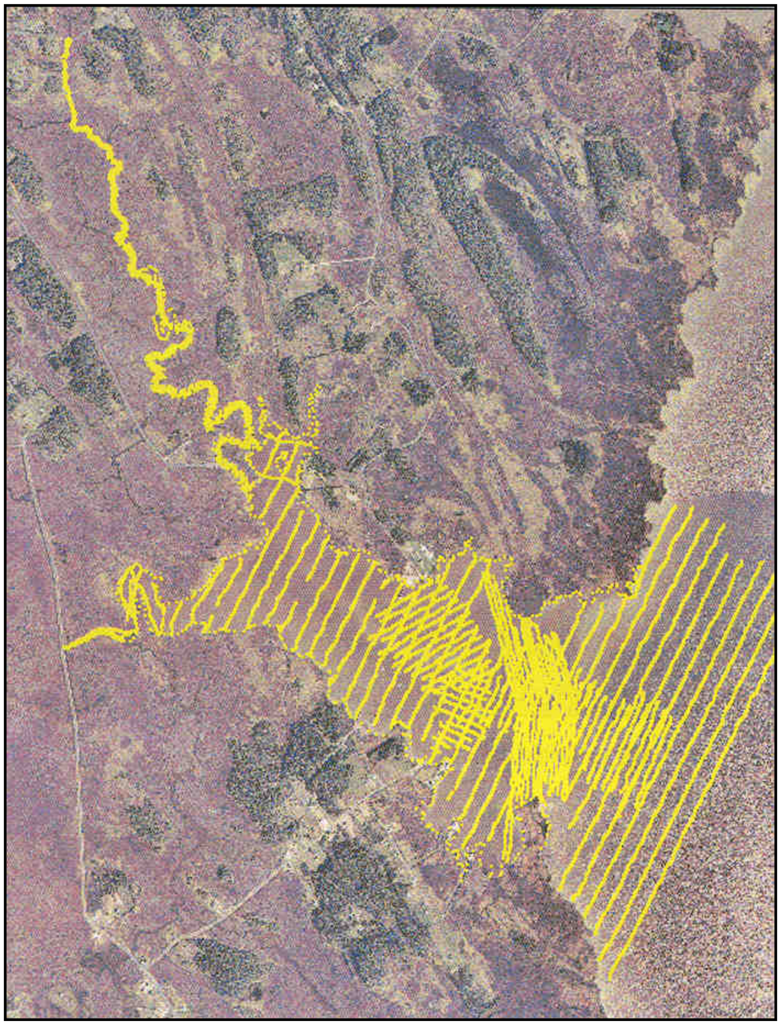


Figure 20. Hydrographic survey lines, Tedious Creek, MD (after Pratt 2003).

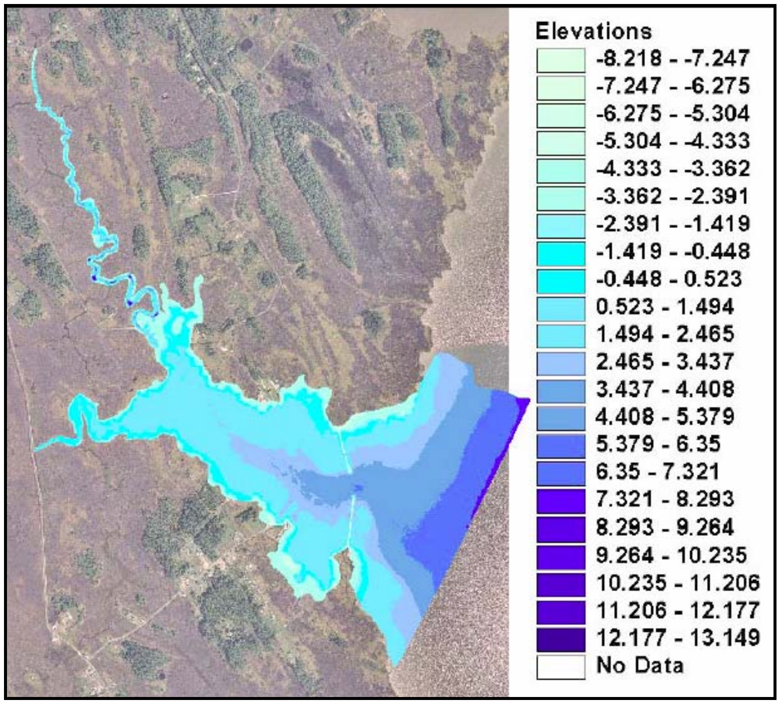


Figure 21. Bathymetric survey contours (ft, mllw), Tedious Creek, MD, August 2001 (after Pratt 2003).

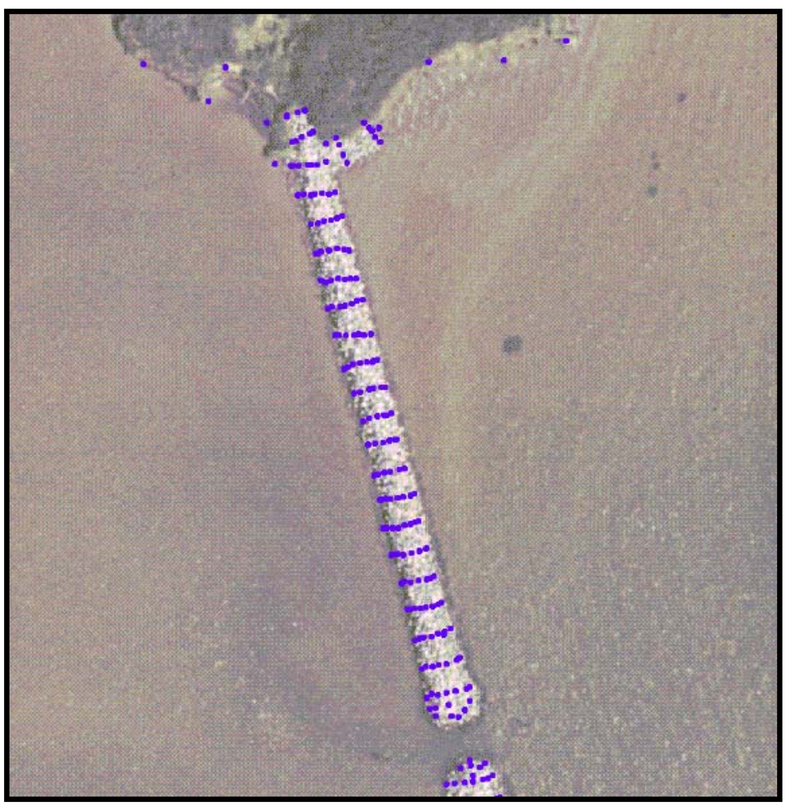


Figure 22. Breakwater cross-section survey lines, and examples of bank line delineation, Tedious Creek, MD (after Pratt 2003).

Breakwater surveys

Surveying the breakwater was an important part of this project. There were concerns that the jetties were settling. The ability to return to the jetty at a later date and survey the same point on each stone was important. This would be possible by using the GPS program and a handheld computer (Figure 23), but a visual method was also needed. Paint might not survive as long as necessary, so construction adhesive was used at each measurement location. Everywhere a data point was collected, a “blob” of adhesive was placed and then a circle was painted around the point. On future surveys, it was possible to return to the exact point and collect comparison data. Data were collected along the entire length of both jetties with cross sections every 6.1 m (20 ft) as shown in Figures 24 and 25. *(Editor's note: Because both the north and south breakwaters at Tedious Creek, MD, each had a gap about midway of their lengths, the four different sections are interchangeably referred to as “jetty” sections.)*

Figure 23. Breakwater survey team using RTK-GPS equipment, with construction adhesive and paint as stone marker points for repetitive surveying (after Pratt 2003).

Figure 24. Tedious Creek, MD, breakwater nomenclature.

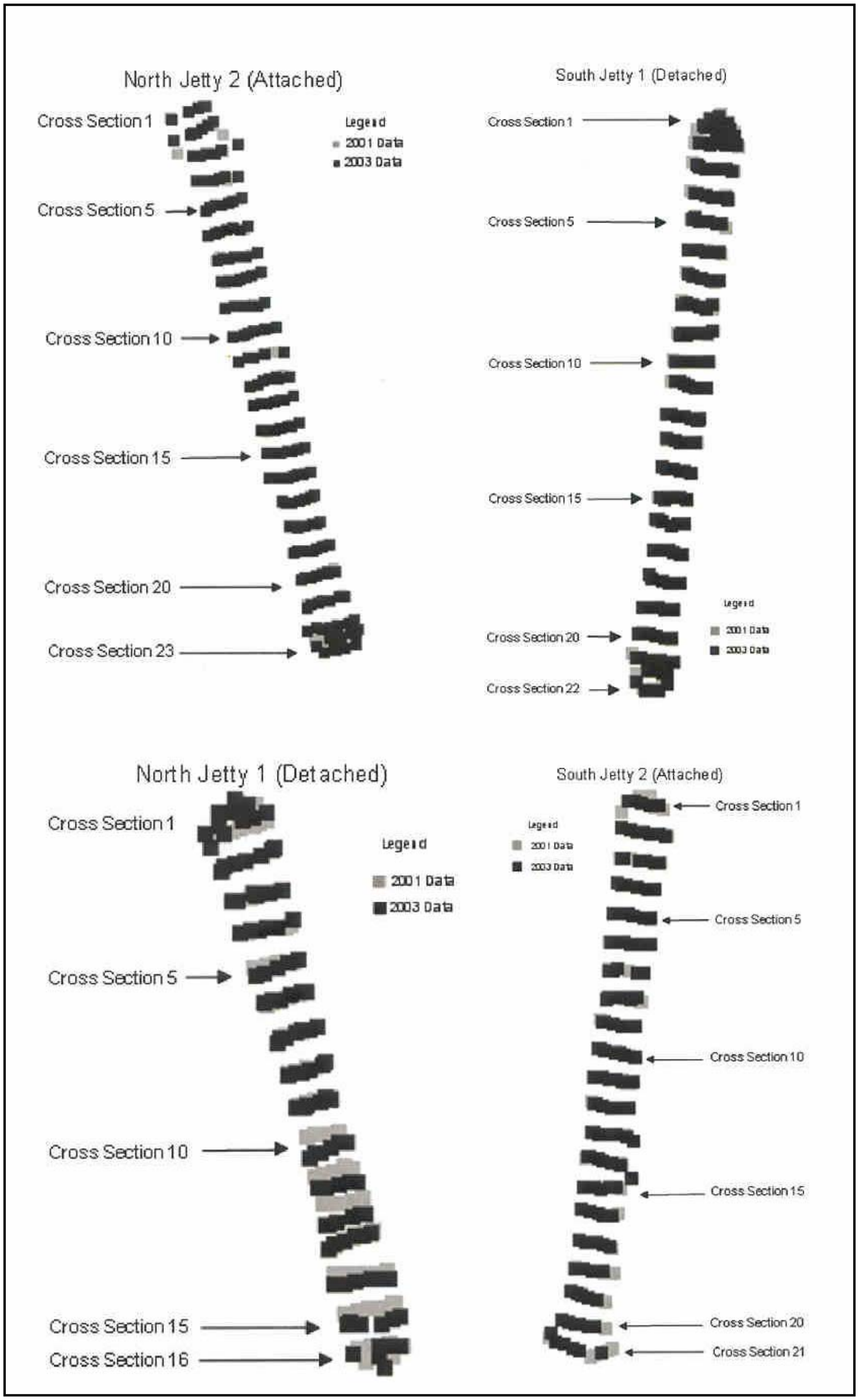


Figure 25. Tedious Creek, MD, breakwater survey cross sections for the August 2001 and August 2003 surveys.

The breakwater surveys conducted in August 2001 and August 2003 measured elevations along cross sections across the breakwater. The surveys were conducted to determine if there had been any changes in the elevations of the breakwater sections during the interval between the two surveys. For all the cross sections, the average change in elevation between the two surveys ranged from 0.04 to 0.09 m (0.13 to 0.28 ft), with a maximum of 0.20 m (0.67 ft). These are distances in a vertical downward direction, indicating settling of jetty stones. (During the data processing, any changes in elevation between 2001 and 2003 that showed the jetty had increased in elevation were considered not applicable (n/a) errors, and were excluded from the statistical analysis.) The average settlement at each of the breakwater (jetty) cross sections is given in Table 8.

Table 8. Average change in elevation between August 2001 and August 2003 breakwater (jetty) surveys, Tedious Creek, MD.

Cross Sec.	Average Change in Elevation, ft			
	North Jetty 1	North Jetty 2	South Jetty 1	South Jetty 2
1	0.33	0.12	0.12	0.16
2	0.17	0.07	0.13	0.18
3	0.15	0.13	0.17	0.16
4	0.15	0.05	0.15	0.15
5	0.64	0.06	0.26	0.20
6	n/a	0.12	0.25	0.17
7	0.23	0.05	0.19	0.06
8	0.26	0.03	0.21	0.09
9	n/a	0.10	0.10	0.24
10	n/a	0.14	0.18	0.17
11	n/a	0.07	0.18	0.33
12	n/a	0.03	0.14	0.14
13	0.25	0.08	0.16	0.20
14	n/a	0.08	0.13	0.38
15	n/a	0.12	0.11	0.10
16	0.34	0.13	0.16	0.23
17	n/a	0.15	0.15	0.62
18	n/a	0.21	0.14	0.11
19	n/a	0.17	0.13	0.14
20	n/a	0.42	0.67	0.15
21	n/a	0.18	0.18	0.11
22	n/a	0.21	0.12	n/a
23	n/a	0.18	n/a	n/a
Min	0.15	0.03	0.10	0.06
Max	0.64	0.42	0.67	0.62
Ave	0.28	0.13	0.18	0.19

Bottom sample collection

Bottom samples were a significant part of the fieldwork on the Tedious Creek MCNP project. Sixty-four bottom samples were collected using a clamshell sampler (Figure 26).

Figure 26. Clamshell bed material sampler used at Tedious Creek, MD (after Pratt 2003).

The samples were stored in 4.4-L (1-gal) ziplock freezer bags for shipment back to the laboratory for grain-size analysis. Standard sieve analysis was performed on these samples. The sample locations were scattered throughout the project area to define the spatial variability of sediment types. Figure 27 shows the layout of the sampling plan. The sediment sieve analysis was summarized and imported into SMS to show the color-coded percent of sediment finer than the sand-cutoff sieve size of 0.63 mm. The two outliers were samples 9 and 56. For sediment sample 56, 100 percent of the material was finer than 2.38-mm sieve size, and 7.49 percent of the sample was finer than 0.063-mm sieve size. For sediment sample 9, 100 percent of the material was finer than 1.00-mm sieve size, and 95.34 percent of the sample was finer than 0.063-mm sieve size. The sediment data were imported into the HyPAS geographic information system (GIS) for analysis and plotting. The sediment toolbox was used to generate the coarse fraction of the gradation curves seen in Figure 28. This toolbox

allows the user to look at all the samples in relation to other data types (bathymetry, velocity, and aerial photography).

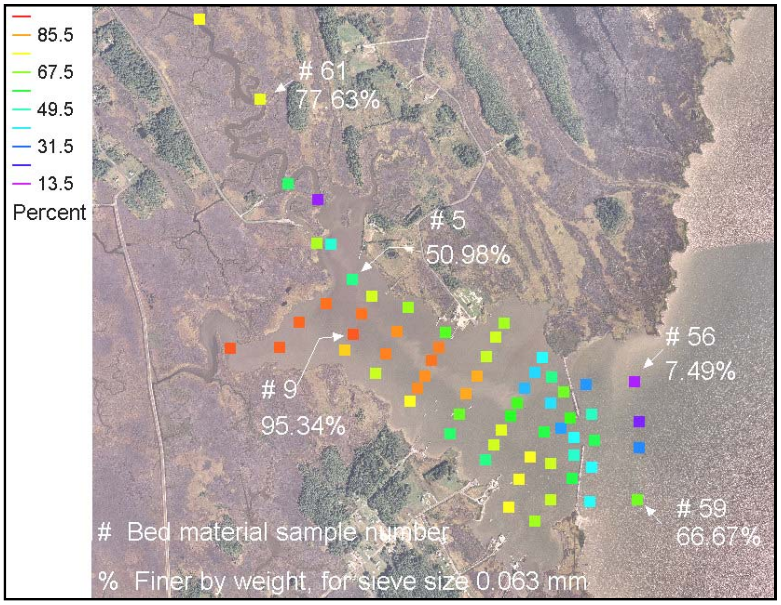


Figure 27. Bed material sampling plan, Tedious Creek, MD.

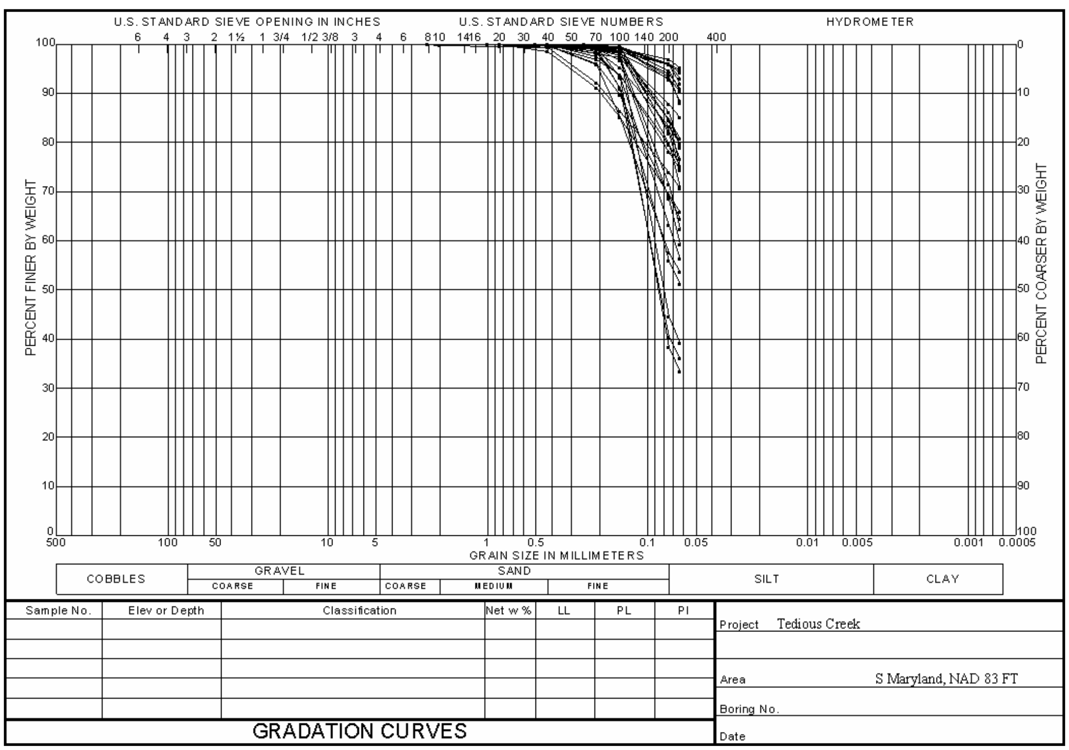


Figure 28. Summary of gradation curves generated from grain-size sieve analyses of bottom sediments, Tedious Creek, MD, showing range of grain sizes throughout the estuary.

Current velocities

The Acoustic Doppler Current Profiler (ADCP) survey involved running three separate lines (1, 2, and 3) in the area of interest, as shown in Figure 29. Each line was run one time each hour for the 13-hr time period. The purpose of this effort was to capture the flow conditions throughout a tidal cycle at critical points in the system. Numerical modelers require the total discharge into the system from Tedious Creek and Fishing Bay. The centerline was located so as to capture eddies generated from the flood jet as it entered the small bay. All of the velocity data for each of the 13-hr ADCP data sets were imported into HyPAS for plotting displays, and for RMA2 hydrodynamic numerical model comparisons. RMA2 is a two-dimensional, depth-averaged finite element hydrodynamic model that computes water surface elevations and horizontal velocity components for subcritical, free-surface flow.

HyPAS also allows for a numerical model solution file data to be imported into the GIS project. Then both model data and ADCP data can be plotted in the same coordinate system using the same legend scales. This is exceedingly useful in making detailed comparisons of these ADCP data types to model results. Since the ADCP data are three-dimensional in nature, and usually of a much greater resolution than model data, they should be processed to match the same vertical and horizontal scales as the model data before a comparison is made. HyPAS affords the user the option of horizontal and vertical averaging at fixed spatial values as specified by the user. Figure 30 shows one time step taken from the hydrodynamic model RMA2 solution file, and the corresponding data set from the 13-hr ADCP survey.

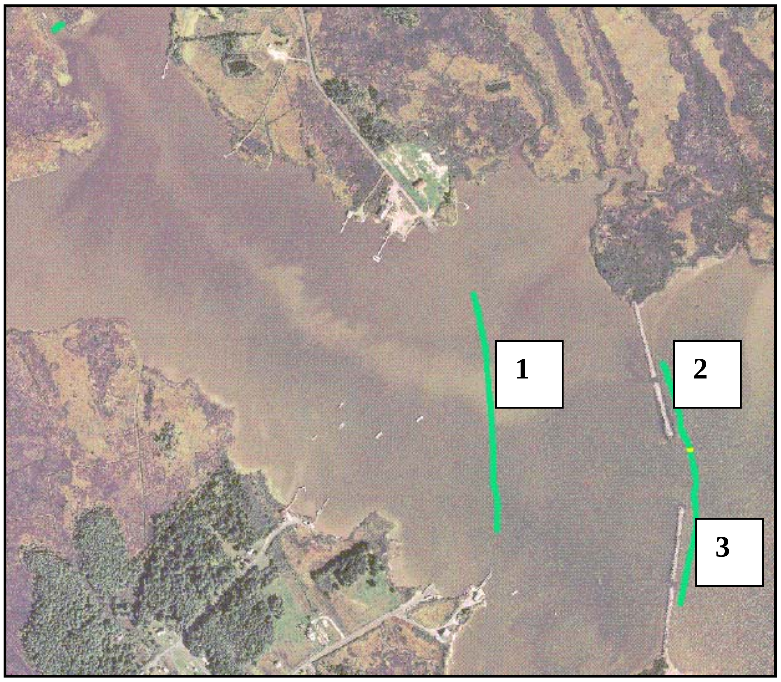


Figure 29. ADCP survey lines, Tedious Creek, MD, August 2001 and September 2002 (after Pratt 2003).

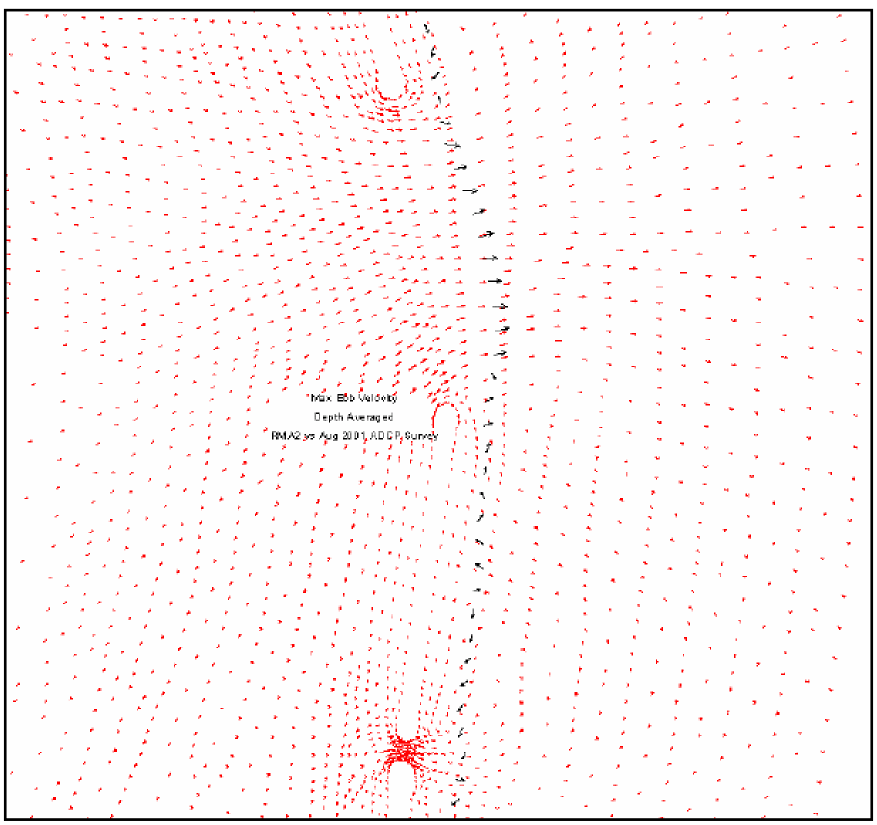


Figure 30. ADCP field data (black vectors) and preliminary RMA2 hydrodynamic numerical model solution (red vectors), Tedious Creek, MD (after Pratt 2003).

In Figure 30, the red vectors are from the numerical hydrodynamic model RMA2 results while the black vectors are from the ADCP field data at a time period representing maximum ebb flow at the center gap. The identify feature in ArcView allows the user to select both vectors when the themes are active. All of the descriptive information defining the vectors is displayed for the user. With comparisons like this, the modeler has the information needed to make necessary adjustments to calibrate and verify the model to reproduce the field data. For additional information on RMA2 verification and results, see Chapter 5.

ADCP currents transect surveys were conducted in August 2001, September 2002, and August 2003. The surveys in 2001 and 2002 were conducted over the three lines (1, 2, and 3) shown in Figure 29. However, the August 2003 survey was conducted over the two lines (2 and 3) shown on the eastern and western ends of the study area, and three additional lines were surveyed. One was along and just inside to the west of the jetty. The other two additional lines were along the jetty, then through the main channel in the jetty, and then along the other side of the jetty. These two lines alternated between being just inside the two southern jetties and just outside the two northern jetties, and being just inside the two northern jetties and just outside the two southern jetties. The standard practice for the surveys was to survey along each line one time each hour for approximately 13 hr. Figure 31 shows the depth-averaged flood currents measured during the August 2003 ADCP survey in the main channel between the north and south jetties. The vector within the small box in the upper left hand corner is the comparable depth-averaged vector taken at the Tedious Creek transect at the back of the embayment. Similarly, Figure 32 shows the ebb currents at the same location.

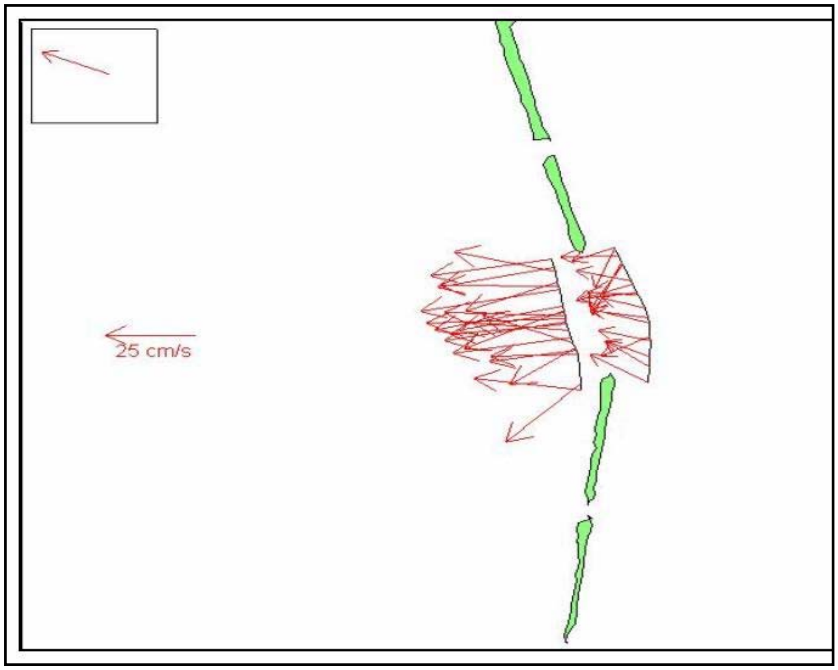


Figure 31. Depth-averaged flood currents derived from ADCP transects, August 2003.

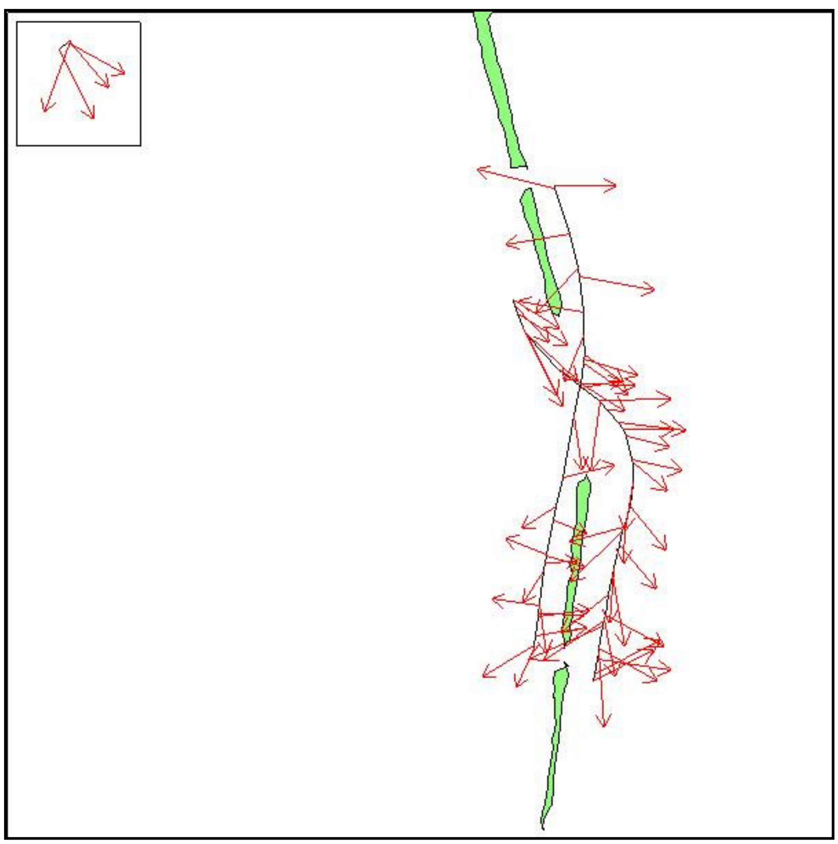


Figure 32. Depth-averaged ebb currents derived from ADCP transects, August 2003 (scale same as Figure 31).

Summary

Water surface elevations, bathymetry, topography, waves, current velocity, and bottom sediment data were collected at the Tedious Creek, MD, small craft harbor between August 2001 and August 2003.The field methods used during the data collection effort are described by Pratt (2003). The data were collected to help evaluate the design and performance of the breakwater constructed at the harbor in 1997.

The water surface elevations and current velocity data were used to calibrate the RMA2 hydrodynamic numerical model of the harbor.

Bathymetry data were used to construct the numerical model mesh and to check for scour and shoaling during the 3 years the study was conducted. No significant changes in bathymetry were noted.

The topographic data consisted of breakwater elevations, and elevations and configuration of the shoreline. No significant changes in the shoreline were detected. Changes in the breakwater elevations were analyzed.

Wave data were collected for approximately 70 days during spring 2003, and were used to calibrate a numerical wave model, Coastal Gravity Wave (CGWAVE), of the area.

4 CGWAVE Numerical Model Comparisons Between Existing As-Built and Authorized Breakwater Configurations[1]

Background

In 1994, the Baltimore District and ERDC CHL conducted hydrodynamic model investigations to determine if a proposed breakwater at Tedious Creek, MD, would have any adverse navigation or environmental effects on the harbor or estuary. The RMA2 hydrodynamic numerical model within the TABS-MD system was used in the design to optimize the gap width and breakwater alignments. RMA2 is a two-dimensional, depth-averaged finite element hydrodynamic model that computes water surface elevations and horizontal velocity components for subcritical, free-surface flow fields. RMA2 computes a finite element solution of the Reynolds form of the Navier-Stokes equations for turbulent flows. Friction was calculated with the Manning equation, and eddy viscosity coefficients were used to define turbulent characteristics. Both steady- and unsteady-state (dynamic) problems can be analyzed. RMA2 was an appropriate tool to test hydrodynamic effects from proposed breakwater locations at Tedious Creek.

Tedious Creek estuary provides anchorage to more than 100 commercial and recreational vessels. The primary anchorages are the county boat dock and the public piers, both on the south shore. Because of its orientation, storm waves cause substantial damage within the estuary. The Baltimore District prepared a Section 107 feasibility report and integrated environmental assessment suggesting that a breakwater could provide a cost-effective solution to the storm wave damage problem (U.S. Army Engineer District, Baltimore 1995). The authorized project design included a breakwater gap of 91 m (300 ft) for the main channel. An additional gap was included midway along both north and south sections of the breakwater to improve water circulation and quality.

[1] This chapter is extracted essentially verbatim from Briggs et al. (2003), with pertinent extractions from Briggs et al. (2005).

The existing as-built breakwater constructed in 1997 differed in geometry from the plans tested in 1994 because of foundation problems. As a result, the north breakwater section is shorter than originally planned, with a 30-m- (100-ft-) wider gap (i.e., 122-m (400-ft) gap) in the main entrance. Because of local concerns, a monitoring effort was initiated in 2001 to test the hypotheses that (a) the existing as-built gap will provide a functional harbor from the standpoint of wave attenuation, circulation, sedimentation, and wetland impacts, (b) the 1997 improvements are structurally sound, (c) the numerical model accurately predicted prototype performance, (d) navigation and the environment will not be adversely impacted by sedimentation from the improvements, and (e) local wetland areas are not adversely impacted. Wave data were collected as part of the MCNP program to calibrate the numerical model CGWAVE for making comparisons between existing and authorized breakwater effectiveness.

CGWAVE model

CGWAVE is a general purpose, state-of-the-art wave prediction model based on the mild slope equation that is used to model waves in harbors, open coasts, inlets, and around islands and fixed and floating structures. It includes (a) effects of wave refraction and diffraction, (b) dissipation from bottom friction, wave breaking, and nonlinear amplitude dispersion, and (c) harbor entrance losses. CGWAVE is a finite element model that is interfaced with the SMS for graphics and efficient implementation (pre- and post-processing). Details for using SMS in a CGWAVE application at Tedious Creek can be found in Appendix A.

Figure 33 shows the orientation of the CGWAVE model domain relative to Tedious Creek and the adjacent countryside. The brown line represents the model boundaries for the harbor and the blue line is the ocean or seaward water boundary. CGWAVE requires a minimum of 6 to 10 elements per wavelength. Since the harbor area is large and shallow and wave periods as small as 6 sec are prevalent, the required grid size was determined to be 1.9 m (6.1 ft). The model size was limited to cover only the most critical areas inside and outside the harbor. The area within the brown boundaries is more than adequate to model wave conditions inside the harbor.

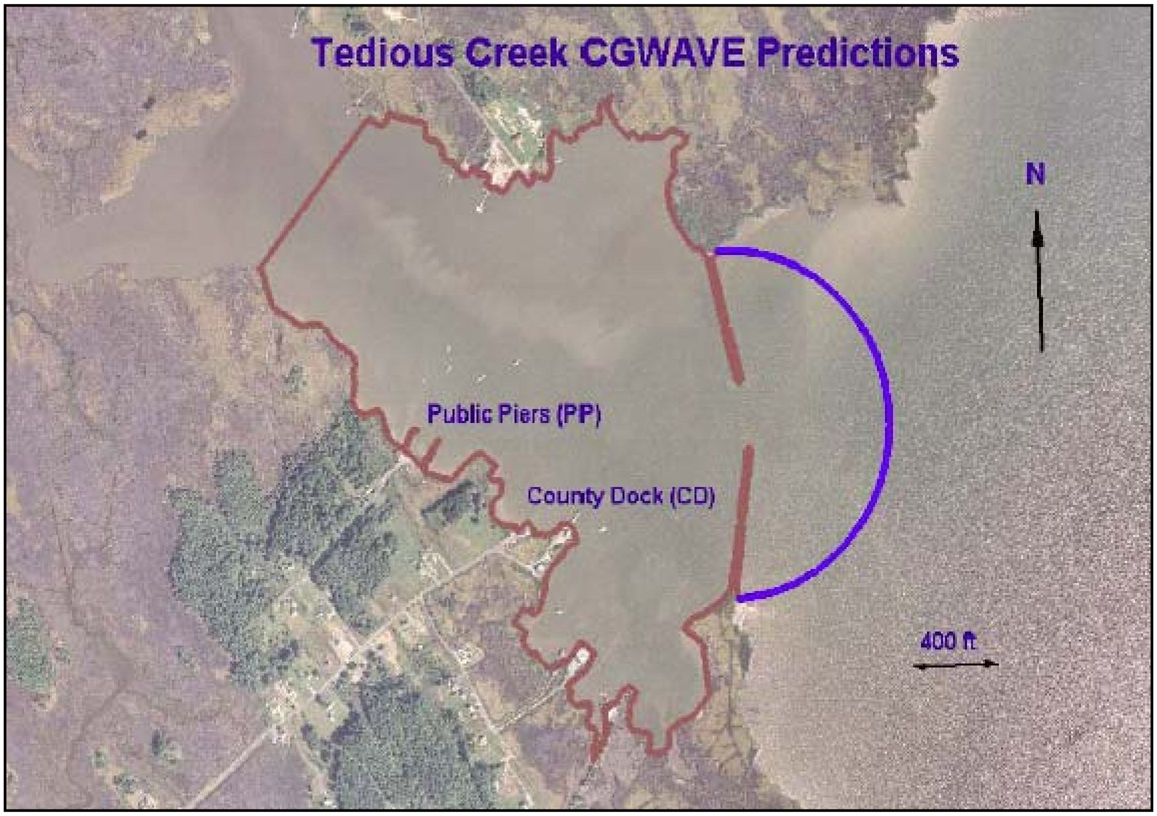


Figure 33. CGWAVE numerical model for Tedious Creek, MD, with existing as-built breakwater with gap of 122 m (400 ft).

Bathymetric data were collected in August 2001 to provide an accurate baseline grid for the modeled area inside and offshore of the breakwaters. The entire Tedious Creek estuary is shallow, with depths less than 2.7 m (8.9 ft). The area outside the ocean boundary was represented by contours from bathymetric charts of the area. These bathymetric contours were used to model the one-dimensional bathymetric lines offshore of the breakwaters required by CGWAVE. Figure 34 shows a contour plot of water depth for a representative storm water level of 1.0 m (3.3 ft) mllw in the existing configuration.

Mean and spring tides range from 0.7 m (2.4 ft) to 0.9 m (3.0 ft), respectively. The 5-year water level, including tide and storm effects, is 1.1 m (3.7 ft) above mllw (U.S. Army Engineer District, Baltimore 1995). A water level of 1.0 m (3.3 ft) mllw was selected as a representative worst case of normally occurring tide and storm levels at Tedious Creek. Since storm wave heights are directly related to water level at this shallow site, only the 1.0-m (3.3-ft) water level was modeled.

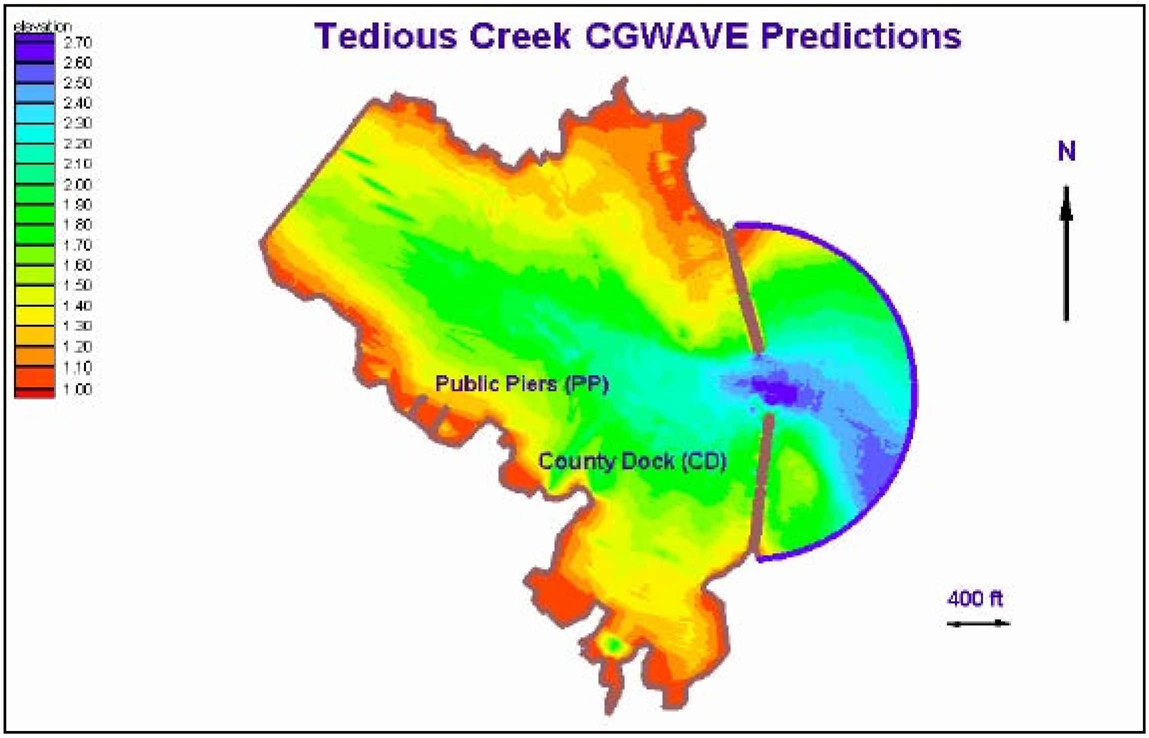


Figure 34. Tedious Creek bathymetry with water level of 1.0 m (3.3 ft) mllw for existing as-built conditions.

Two different model grids were created in CGWAVE. The existing as-built model included a breakwater gap of 122 m (400 ft). The authorized project model has a breakwater gap of 91 m (300 ft).

Nine different wave conditions, composed of three wave periods from three wave directions each, were selected for study based on harbor orientation, limited field measurements, local observations, and previous analysis (Offshore and Coastal Technologies Incorporated 2001). Wave periods T = 6, 10, and 16 sec were selected as representative of the range of wave periods occurring in the harbor. The Offshore and Coastal Technologies Incorporated (2001) study used a JONSWAP spectrum with a peak period in the range of 8 to 10 sec. A value of T = 6 sec was chosen as the minimum period for use by CGWAVE. The T = 16-sec value was selected as the worst-case swell waves that might propagate into the harbor from Atlantic storms. Mean wave directions of θ = 135, 180, and 225 deg represent waves propagating to the northwest, west, and southwest (angles measured counterclockwise from the east), respectively. The Offshore and Coastal Technologies Incorporated (2001) study used these direction limits, with waves from the southwest considered the worst case for the county boat dock even though they also tested waves from 210 deg

(west-southwest). Waves traveling to the west and southwest are most likely to affect the public pier and the county boat dock since they have a clear path through the breakwater gap. According to the Section 107 feasibility report of 1995, 5-year storm significant wave heights arriving from the northeast through southeast quadrant range from H = 0.5 m (1.6 ft) to 0.7 m (2.3 ft). Fifty-year storm heights vary from H = 0.6 m (2.1 ft) to 1.5 m (4.9 ft). A representative value of 1 m (3.3 ft) was selected as the incident wave height H_i at the offshore grid boundary. All modeled waves were regular (i.e., monochromatic).

The CGWAVE model includes wave reflection from solid boundaries. Reflection coefficients of C_r = 0.0, 0.1, 0.5, and 0.9 were selected for the open ocean, inner bay perimeter, rubble-mound breakwater, and public pier, respectively. CGWAVE requires small element sizes in shallow-water regions for accurate description of short-period waves.

CGWAVE sensitivity

Because local interest was in the areas near the county boat dock and the public pier, two transects (T1 and T2) were selected between these two facilities and the breakwater entrance. Figures 35 and 36 show the orientation of these two transects for both existing and authorized model layouts, respectively. Transect T1 is 481 m (1,579 ft) long between the county boat dock and the breakwater entrance, and transect T2 is 673 m (2,207 ft) long between the public pier and the breakwater entrance.

CGWAVE has options to include bottom friction and wave breaking. The first step in the model calibration was to run test cases to quantify their effect on predicted wave heights H inside the harbor. Thus, the T = 6 sec waves traveling to the west (i.e., θ = 180 deg) were run for base cases (a) with no bottom friction or wave breaking (none), (b) with wave breaking only (WB), (c) with bottom friction only (BF), and (d) combined bottom friction plus wave breaking (BFWB).

Figure 37 shows the wave height predictions along transects T1 and T2 for each of the four sensitivity parameter combinations. As expected, wave height was larger for the “none” case (i.e., linear mode) with no bottom friction or wave breaking. Of course, in the linear mode, model predictions would be unrealistic as these two wave phenomena are present in nature and would naturally limit the wave heights. Inclusion of bottom friction was more significant than wave breaking. Even though the difference

between bottom friction and bottom friction plus wave breaking was slight, it was decided to include the combined effects of both phenomena in comparison of the two configurations.

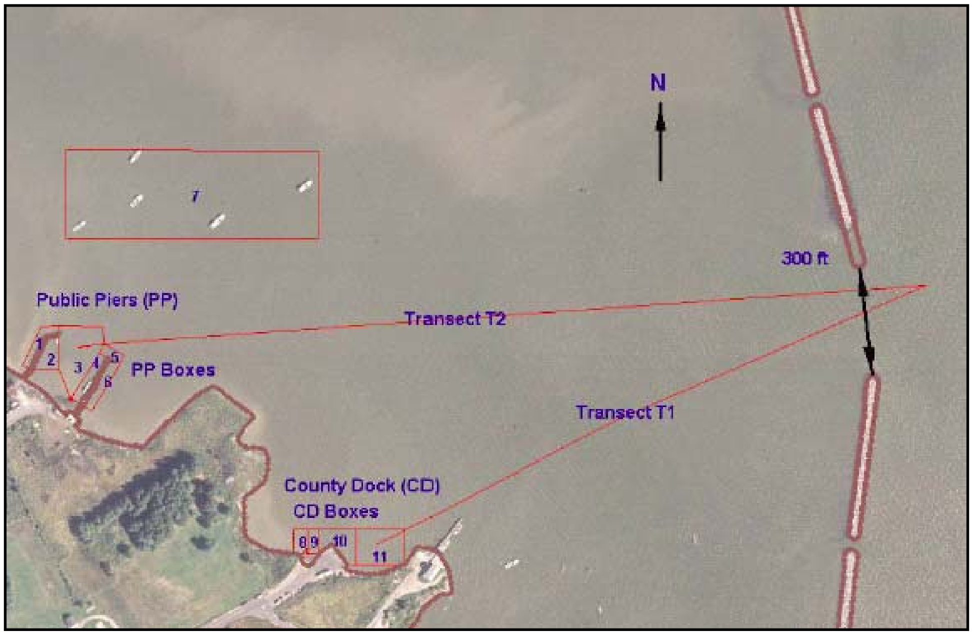


Figure 35. Orientation of transect lines T1 and T2 and 11 computational boxes for existing as-built breakwater with 122-m (400-ft) gap.

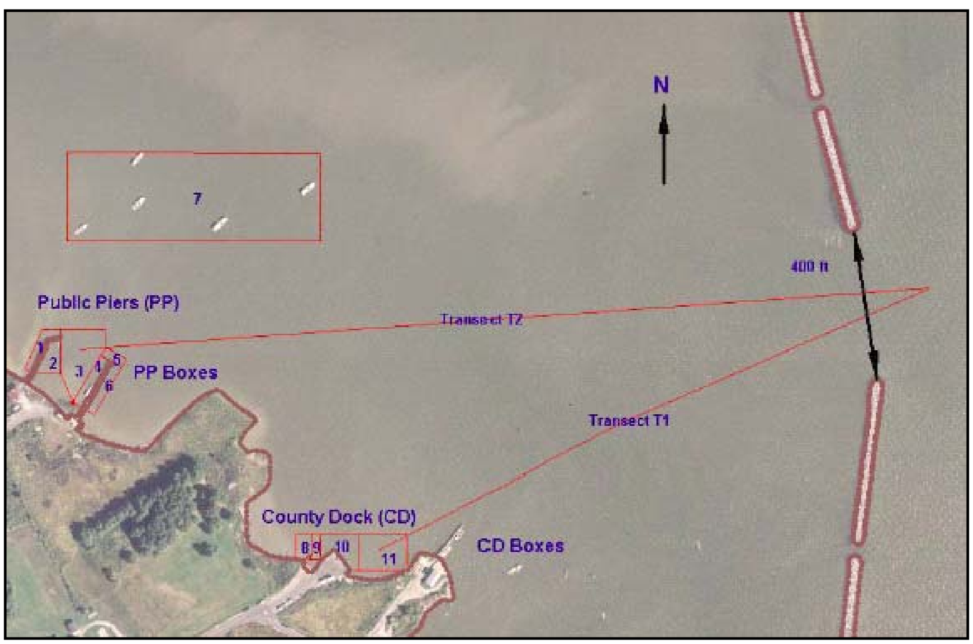


Figure 36. Orientation of transect lines T1 and T2 and 11 computational boxes for authorized breakwater with 91-m (300-ft) gap.

CGWAVE calibration

Field data were acquired between April and July 2001, with three wave gages (Figure 38). A NORTEK Aquadopp directional wave gage (PUV-type) measured incident wave conditions outside the breakwater in an average depth of 2.2 m (7.2 ft) mean low water (mlw). Two unidirectional pressure gages measured transformed wave conditions inside the breakwater in 1.7 m (5.6 ft) mlw (gage 215) and 1.3 m (4.3 ft) mlw (gage 212). Gage 215 lay on transect T1. The sampling frequency was 4 Hz, with a sampling interval every 3 hr, for all three gages.

Figure 39 is a time series of incident wave period, significant wave height, and wave direction. Measured PUV wave directions were converted from wave direction from which waves travel, measured clockwise from north, to CGWAVE conventions. Figure 40 shows transmitted wave heights for gages 212 and 215. Since this was a milder time of the year and there were no major storms, incident wave conditions were fairly benign (i.e., in the range of T = 2 to 5 sec, H_s < 0.5 m, and average wave approach direction of θ = 118 deg or waves traveling to north-northwest. The transmitted wave heights were even smaller, with a maximum of 0.31 m (1.1 ft) and averages of 0.06 m (0.2 ft).

Because these waves were small, it was difficult to compare them to CGWAVE predictions. A few limited comparisons were made and are shown in Figure 41 for the two transmitted gage locations. Measured incident wave parameters ranged from $2.03 \leq T \leq 4.75$ sec, $0.07 \leq H_s \leq 0.27$ m, and $135 \leq \theta \leq 221$ deg. The largest wave period and height combinations were determined. An equivalent significant wave height and exact wave period and direction were used as input to CGWAVE. The model predictions were averaged over a 30-m (100-ft) square box around each field gage location (Figure 38) to allow for location anomalies and the contouring algorithm in the model. Considering that CGWAVE was designed for wave periods on the order of 5 sec or larger, the agreement was very good.

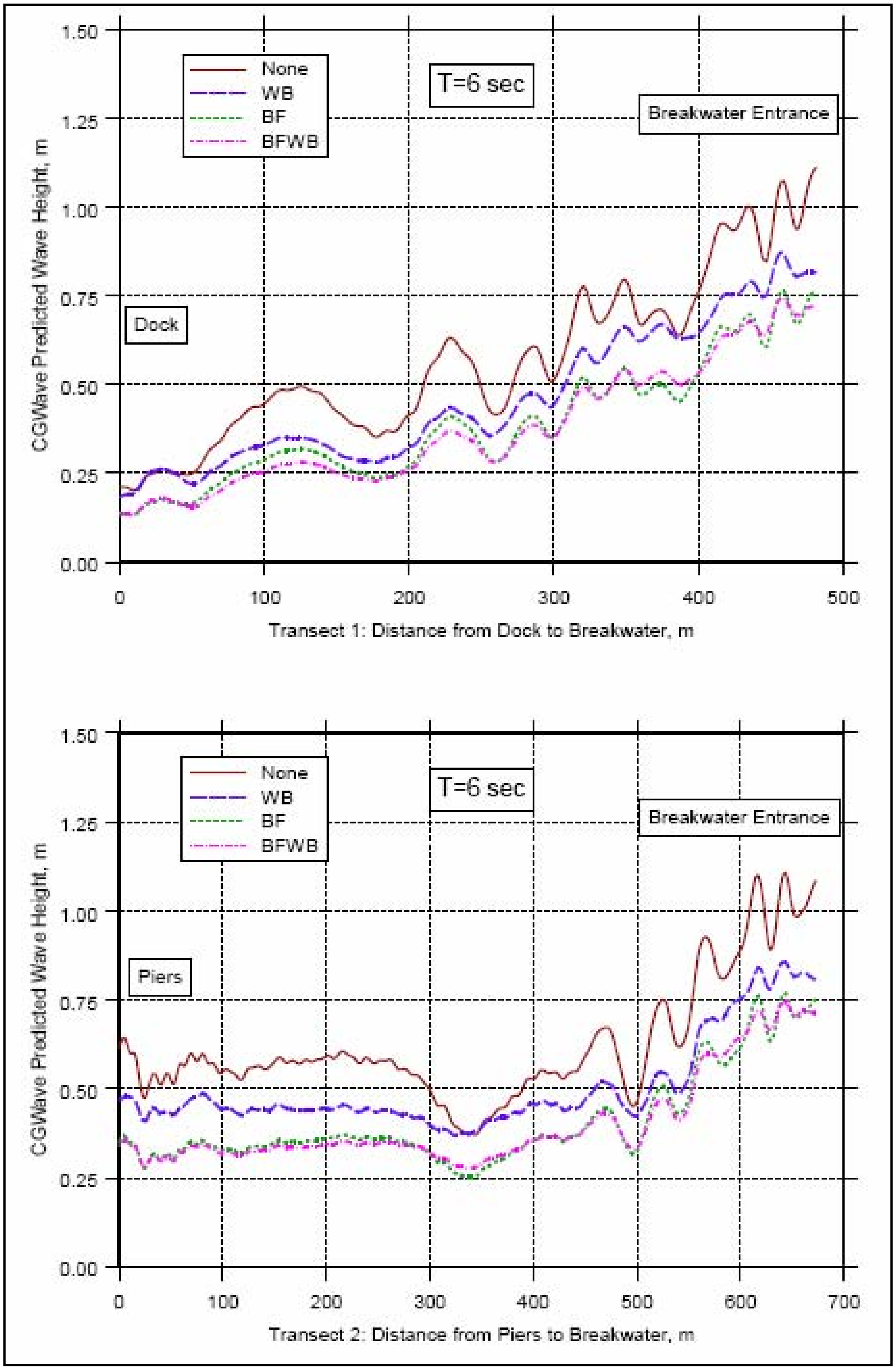


Figure 37. CGWAVE wave heights for no breaking (none), wave breaking (WB), bottom friction (BF), and bottom friction plus wave breaking (BFWB) for T= 6 sec, H_i = 1 m (3.3 ft), water level = 1 m (3.3 ft) mllw, and wave approach direction θ = W (180 deg) along transect T1 and transect T2.

Figure 38. Field wave gage locations, April through July 2001, Tedious Creek, MD.

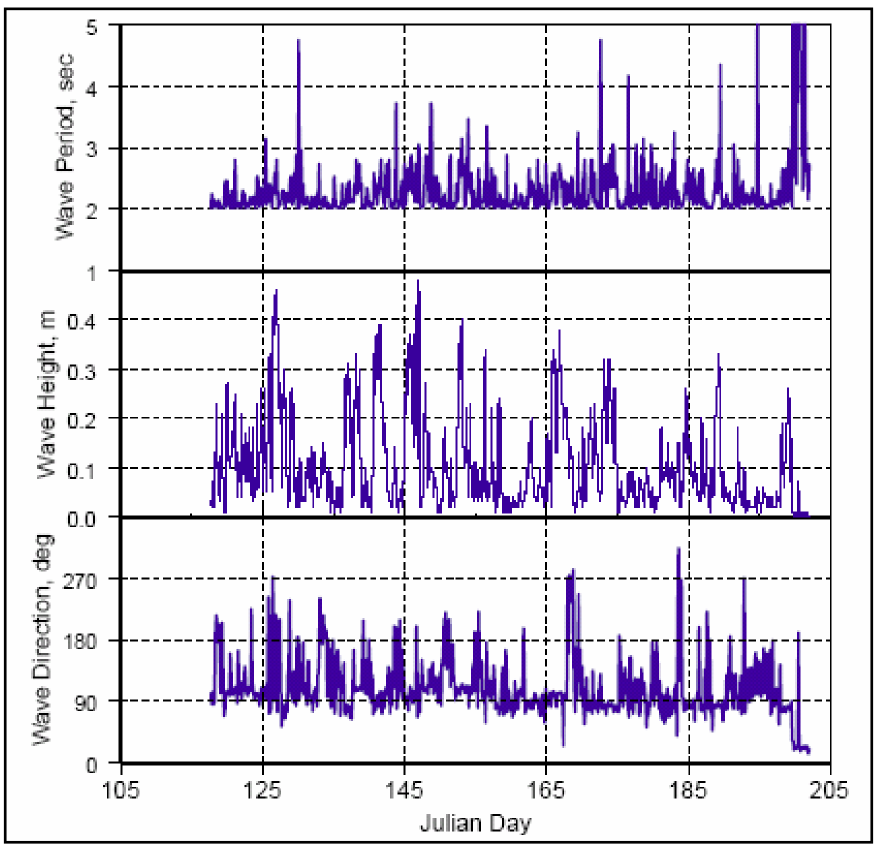


Figure 39. Measured incident wave parameters gage incident PUV outside the breakwater, April through July 2001, Tedious Creek, MD.

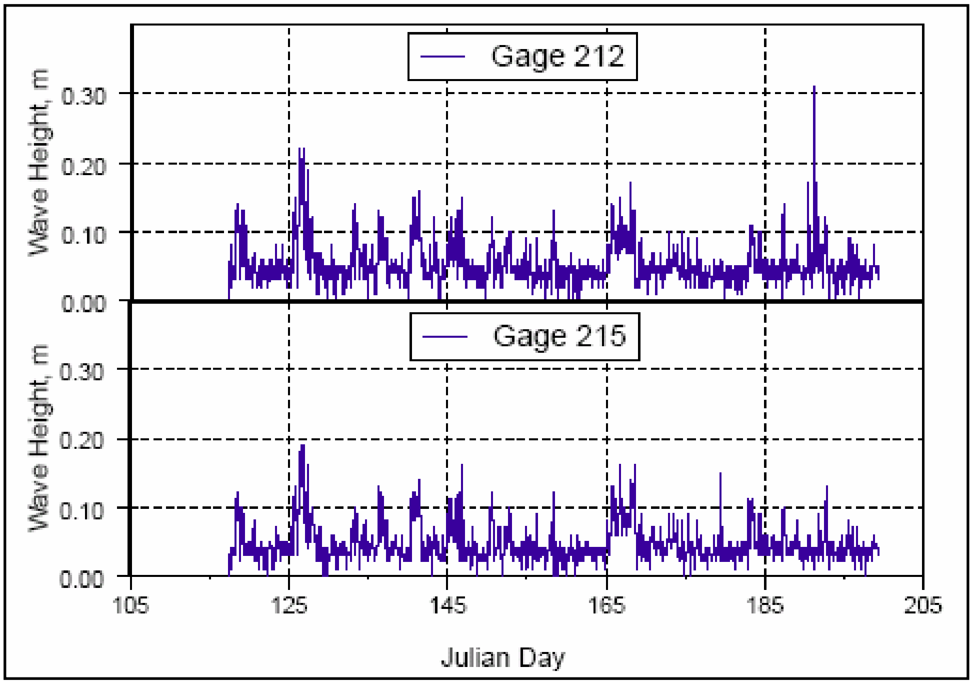


Figure 40. Measured transmitted wave heights at gages 212 and 215, April through July 2001, Tedious Creek, MD.

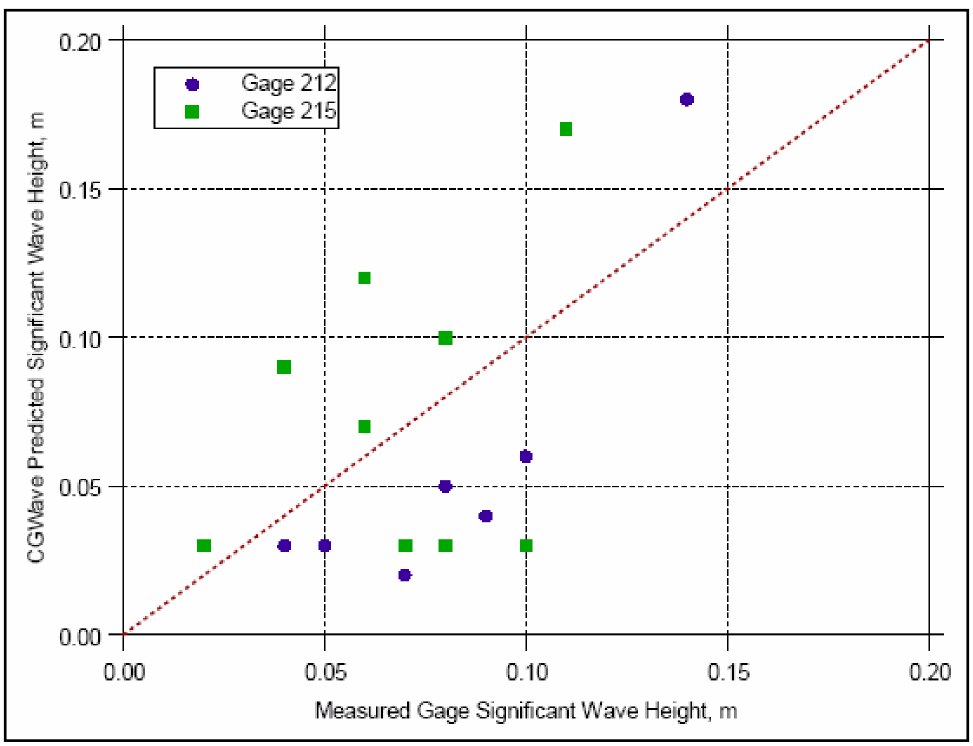


Figure 41. Comparison of CGWAVE-predicted wave heights with measured field site wave heights, April through July 2001, Tedious Creek, MD.

Existing as-built breakwater wave heights

The existing as-built breakwater gap was 122 m (400 ft). Predicted wave heights within the harbor area from the CGWAVE model are presented for nine wave conditions. Because a distribution of historical wave conditions is not available for this location, all wave conditions are assumed to occur with equal probability. Waves with a T = 6 sec, however, are probably the most typical wave conditions due to the shallow depths and the orientation of the harbor within Chesapeake Bay.

Figure 42 is a contour plot of the wave height H for T = 6-sec waves and three wave directions traveling to θ = 135 (NW), 180 (W), and 225 (SW) deg, respectively. The constant-length vectors illustrate wave directions within the domain. The incident wave height is 1.0 m (3.3 ft) on the offshore boundary and decreases as waves propagate into the harbor. Larger wave heights occur to the north for waves traveling to the northwest, to the west and vicinity of the public pier for waves traveling to the west, and to the southwest and vicinity of the county boat dock for waves traveling to the southwest. The wave patterns are similar for the longer wave periods (not shown), and display slightly different penetration patterns of wave energy. Figures 43 and 44 show the predicted wave heights H along transects T1 and T2, respectively, for the three wave periods T = 6, 10, and 16 sec for the existing configuration. The three curves on each plot represent the three different wave directions θ = 135 (NW), 180 (W), and 225 (SW) deg, respectively. For transect T1 going to the county boat dock, waves traveling to the southwest (green, short dash line) are the highest in the inner part of the harbor. Waves traveling to the west (blue, dash line) become higher in the vicinity of the breakwater entrance. The crossover point between these two wave directions moves closer to the breakwater entrance as wave period increases. Waves traveling to the northwest (red, solid line) are smaller since they propagate away from the area of transect T1. For transect T2 going to the public pier, waves traveling to the west are the highest throughout the harbor region for all wave periods. Waves traveling to the other two directions are nearly the same for the first 400 to 500 m (1,312 to 1,640 ft) from the public pier, where the waves traveling to the southwest are higher.

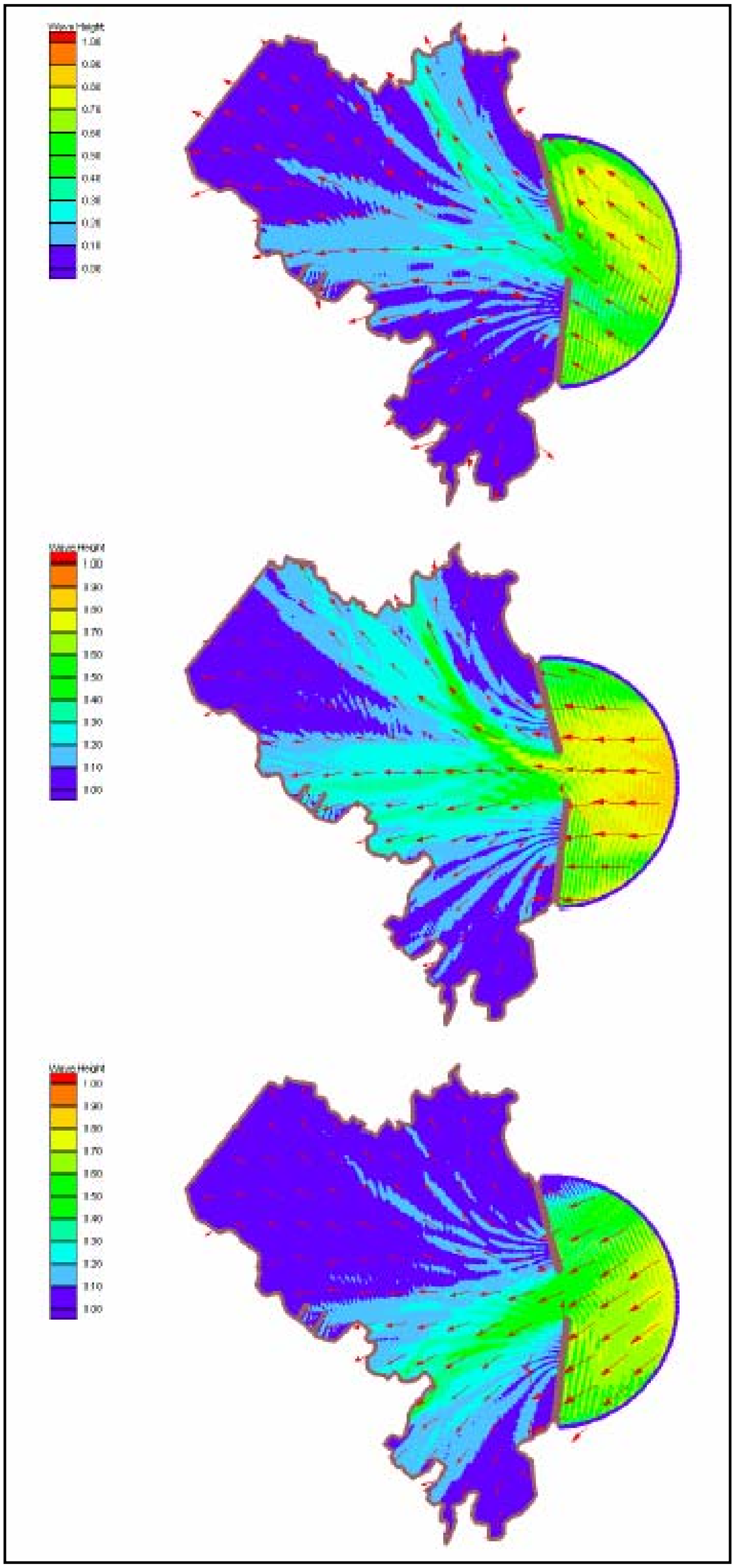


Figure 42. CGWAVE wave height predictions for existing as-built breakwater configuration with wave conditions of T = 6 sec, H_i = 1 m (3.3 ft), water level = 1 m (3.3 ft) mllw, and wave directions (a) θ = 135 deg (NW), (b) θ = 180 deg (W), and (c) θ = 225 deg (SW).

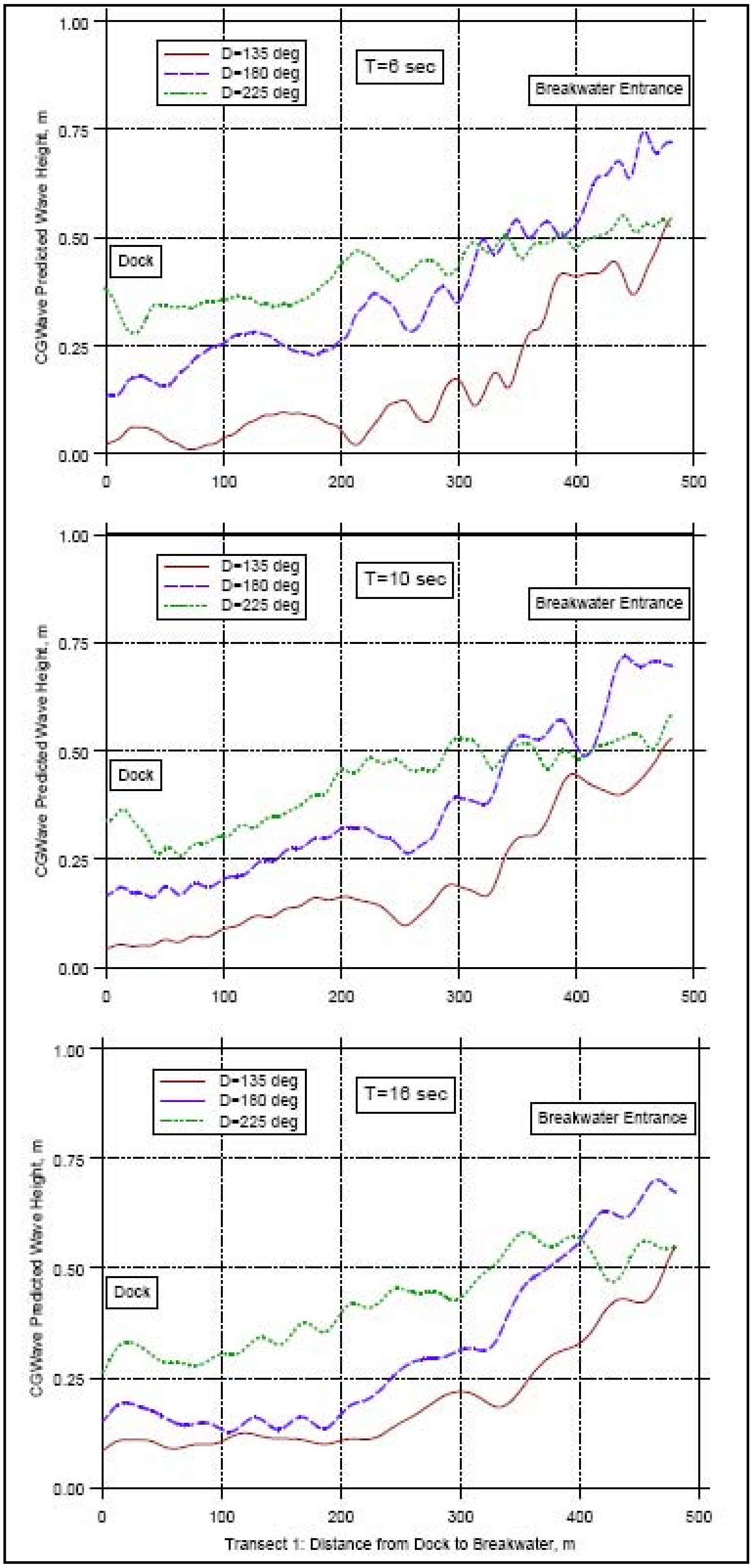


Figure 43. CGWAVE wave height predictions along transect T1 for existing as-built breakwater configuration with wave conditions of H_i = 1 m (3.3 ft), water level = 1 m (3.3 ft) mllw, and wave periods T = 6 sec, T = 10 sec, and T = 16 sec.

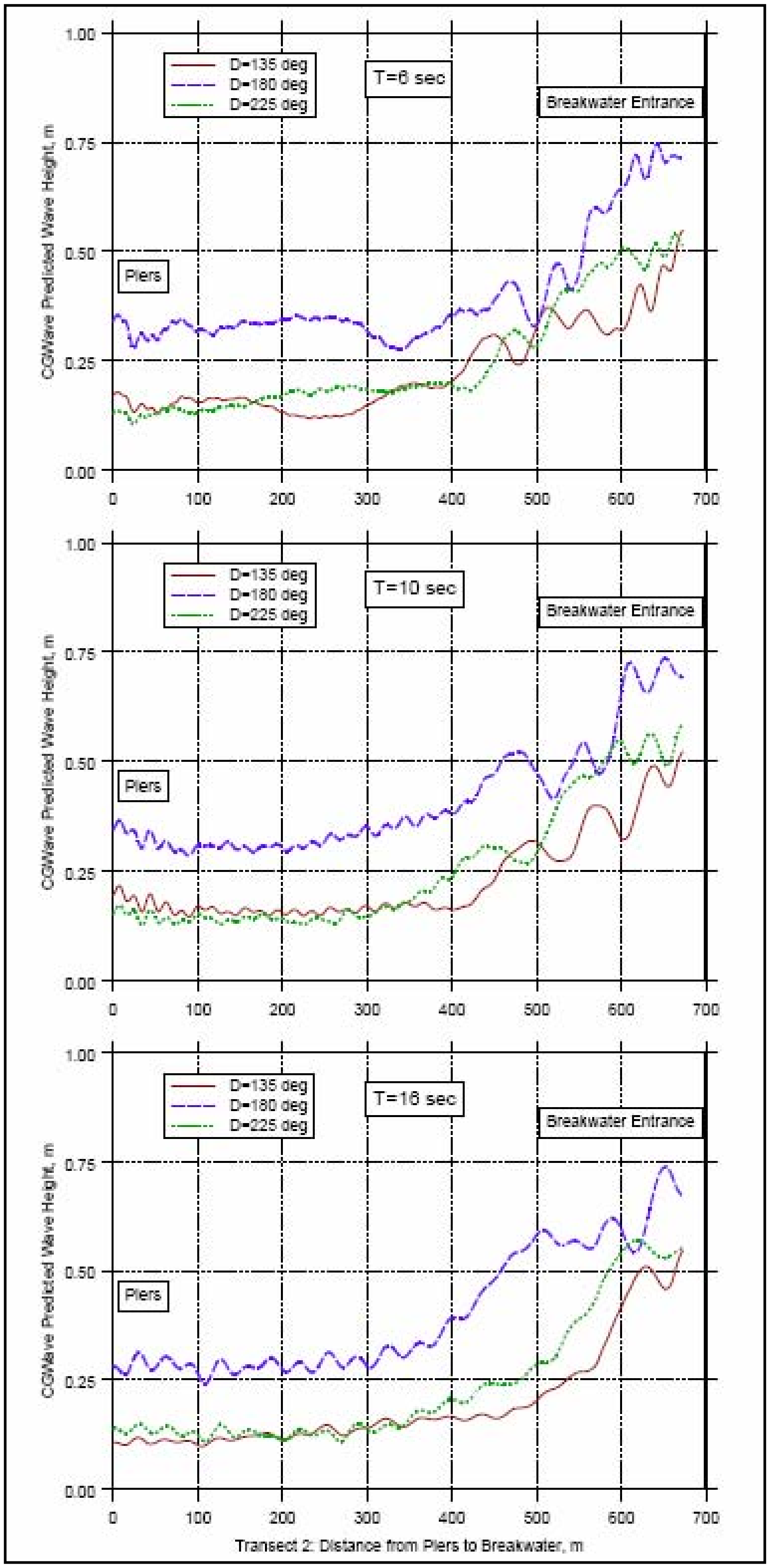


Figure 44. CGWAVE wave height predictions along transect T2 for existing as-built breakwater configuration with wave conditions of H_i = 1 m (3.3 ft), water level = 1 m (3.3 ft) mllw, and wave periods T = 6 sec, T = 10 sec, and T = 16 sec.

Wave height statistics were calculated along transects T1 and T2 to better quantify wave heights. The CGWAVE model predicted 414 wave heights along the 481-m- (1,578-ft-) long transect T1, and 616 along the 672-m- (2,205-ft-) long transect T2. The minimum (H_{Min}), maximum (H_{Max}), average ($\bar{H}$), and standard deviation wave height (H_σ) for transects T1 and T2 are listed for all wave conditions in Table 9. Of course, the H_{Max} occurs near the breakwater entrance in most cases that is outside the protection of the breakwater. Based on the data, the average wave height along transect T1 will be in the range of 0.31 ± 0.02 m (1.02 ± 0.07 ft) with 95-percent confidence. The average wave height along transect T2 will be in the range of 0.28 ± 0.01 m (0.92 ± 0.03 ft) with 95-percent confidence.

Table 9. Wave height statistics for transects T1 and T2.

Parameter	T1	T2
Existing Configuration		
H_{Min}	0.01 m	0.08 m
H_{Max}	0.78 m	0.76 m
$\bar{H}$	0.31 m	0.28 m
H_σ	0.11 m	0.11 m
Authorized Configuration		
H_{Min}	0.01 m	0.03 m
H_{Max}	0.81 m	0.86 m
$\bar{H}$	0.27 m	0.24 m
H_σ	0.09 m	0.12 m

The two transects provide a general overview of wave height within the harbor. However, because of the significant variability in wave height along each one as a function of position and water depth, it is difficult to compare wave energy between the two configurations based on an average wave height along these transects. Another approach was to average the wave heights within limited areas or computational boxes adjacent to the public pier and county boat dock. Each computational box contains many nodes from the CGWAVE model, each with a predicted wave height value. Figure 36 shows a set of seven computational boxes (i.e., boxes 1 to 7) in the vicinity of the public pier, and four computational boxes (i.e., boxes 8 to 11) adjacent to the county boat dock. Boxes 1 and 2 are adjacent to the westerly public pier, boxes 4 to 6 are adjacent to the easterly public pier, box 3 is between the two piers, and box 7 contains the offshore docking area. Boxes 8 and 9 are in the vicinity of the boat ramp of the county boat

dock, and boxes 10 and 11 are to the east of the county boat dock. The end points of transects T1 and T2 can be seen in the interior of boxes 11 and 3, respectively.

Figure 45 shows the average wave heights in each box H for the three wave periods. The public pier boxes (1 to 7) are to the left of the vertical red line, and the county boat dock boxes (8 to 11) are to the right. For the public pier boxes, the highest wave heights occur for waves traveling to the west (θ = 180 deg). For the county boat dock boxes, the highest heights occur for waves traveling predominantly to the southwest (θ = 225 deg).

The maximum wave heights H_{Max} and their box locations from Figure 45 are shown in Table 10 for the boxes in the public pier and county boat dock areas. Averaged over all boxes in the public pier area, the maximum average wave heights H_{All} = 0.23, 0.26, and 0.23 m (0.75, 0.85, and 0.75 ft) for T = 6, 10, and 16 sec, respectively. All maximum values are for waves traveling to the west. Averaged over all boxes in the county boat dock area, the maximum average wave heights H_{All} = 0.26, 0.25, and 0.23 m (0.85, 0.82, and 0.75 ft) for T = 6, 10, and 16 sec, respectively. All maximum values are for waves traveling to the southwest for the county boat dock boxes. Finally, the 95-percent confidence intervals for average wave height inside all boxes for all wave conditions are 0.16 ± 0.02 m (0.52 ± 0.07 ft) in the public pier boxes and 0.16 ± 0.03 m (0.52 ± 0.10 ft) in the county boat dock boxes. In general, the box area wave heights correlate well with the predicted values for the two transects. The average wave heights for the boxes are smaller than the transect averages since they only cover the shallower areas that have experienced greater energy dissipation (e.g., further wave diffraction behind the breakwater, energy losses due to bottom friction and wave breaking, etc.).

Table 10. Maximum average wave heights in box areas.[1]

Parameter	PP Boxes	CD Boxes
Existing Configuration		
T = 6 sec	0.31 m (Box 5)	0.30 m (Box 11)
T = 10 sec	0.37 m (Box 6)	0.29 m (Box 11)
T = 16 sec	0.34 m (Box 6)	0.26 m (Box 10)
Authorized Configuration		
T = 6 sec	0.33 m (Box 5, 6)	0.23 m (Box 8)
T = 10 sec	0.38 m (Box 6)	0.20 m (Box 10)
T = 16 sec	0.30 m (Box 6)	0.21 m (Box 10)
[1] PP = public pier, CD = county boat dock.		

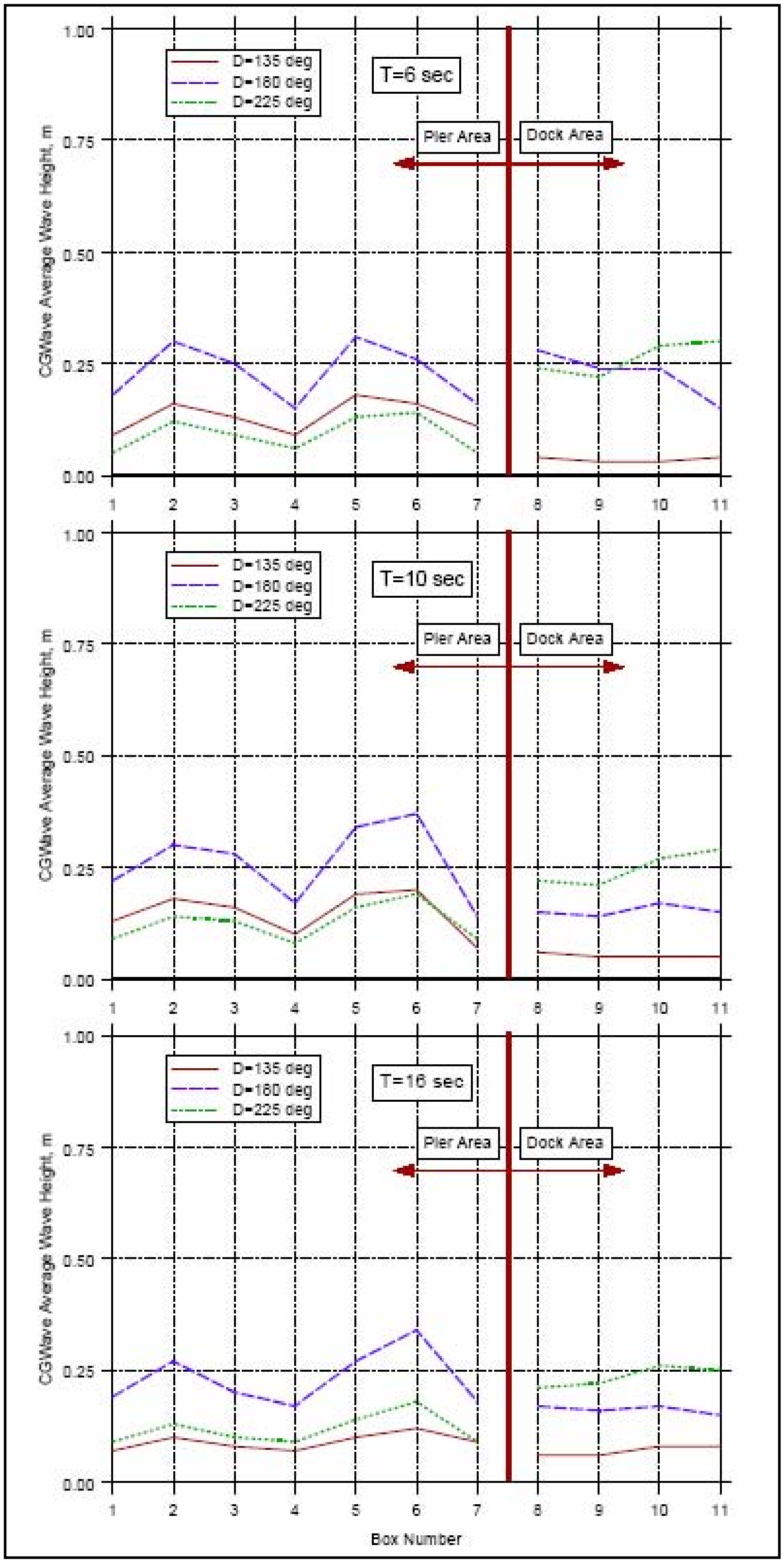


Figure 45. CGWAVE average wave heights in computational boxes for existing as-built breakwater configuration for waves with H_i = 1 m (3.3 ft) mllw, water level of 1 m (3.3 ft), and wave periods T = 6 sec, T = 10 sec, and T = 16 sec.

Authorized breakwater wave heights

The authorized breakwater gap was 91 m (300 ft), or 30 m (100 ft) shorter than the existing breakwater gap. The analysis of the authorized breakwater configuration is similar to the analysis of the existing as-built breakwater configuration. Figure 46 is a contour plot of the wave height H for wave period T = 6-sec waves and three wave directions, similar to Figure 42 for the existing as-built breakwater. The wave patterns for all wave periods are similar to the existing breakwater, except that the smaller gap reduces the extent of the wave incursion into the harbor.

Figures 47 and 48 show the predicted wave heights H along transects T1 and T2, respectively, for the three wave periods T = 6, 10, and 16 sec for the authorized configuration. For transect T1 and waves approaching the county boat dock, the narrower gap significantly reduces wave energy inside the harbor for waves traveling to the southwest (green short dash line). This is in agreement with Offshore and Coastal Technologies Incorporated (2001) findings. Although they had run some tests with waves traveling midway between west and southwest (equivalent to west-southwest or θ = 210 deg), these waves did not prove to be worse than the southwest waves (θ = 225 deg). The largest waves are now traveling to the west (blue, dash line). The difference in wave height for waves traveling to the west and waves traveling to the southwest decreases as wave period increases. For transect T2 and waves going to the public pier, the largest waves are traveling to the west.

The CGWAVE model provided 421 wave heights along the 481-m- (1,578-ft-) long transect T1, and 610 along the 672-m- (2,205-ft-) long transect T2. Wave height statistics are again summarized in Table 9 for all wave conditions along the two transects. Based on the data, the average wave height along transect T1 will be in the range of 0.27 ± 0.01 m (0.89 ± 0.03 ft) with 95-percent confidence. The average wave height along transect T2 will be in the range of 0.24 ± 0.01 m (0.79 ± 0.03 ft) with 95-percent confidence.

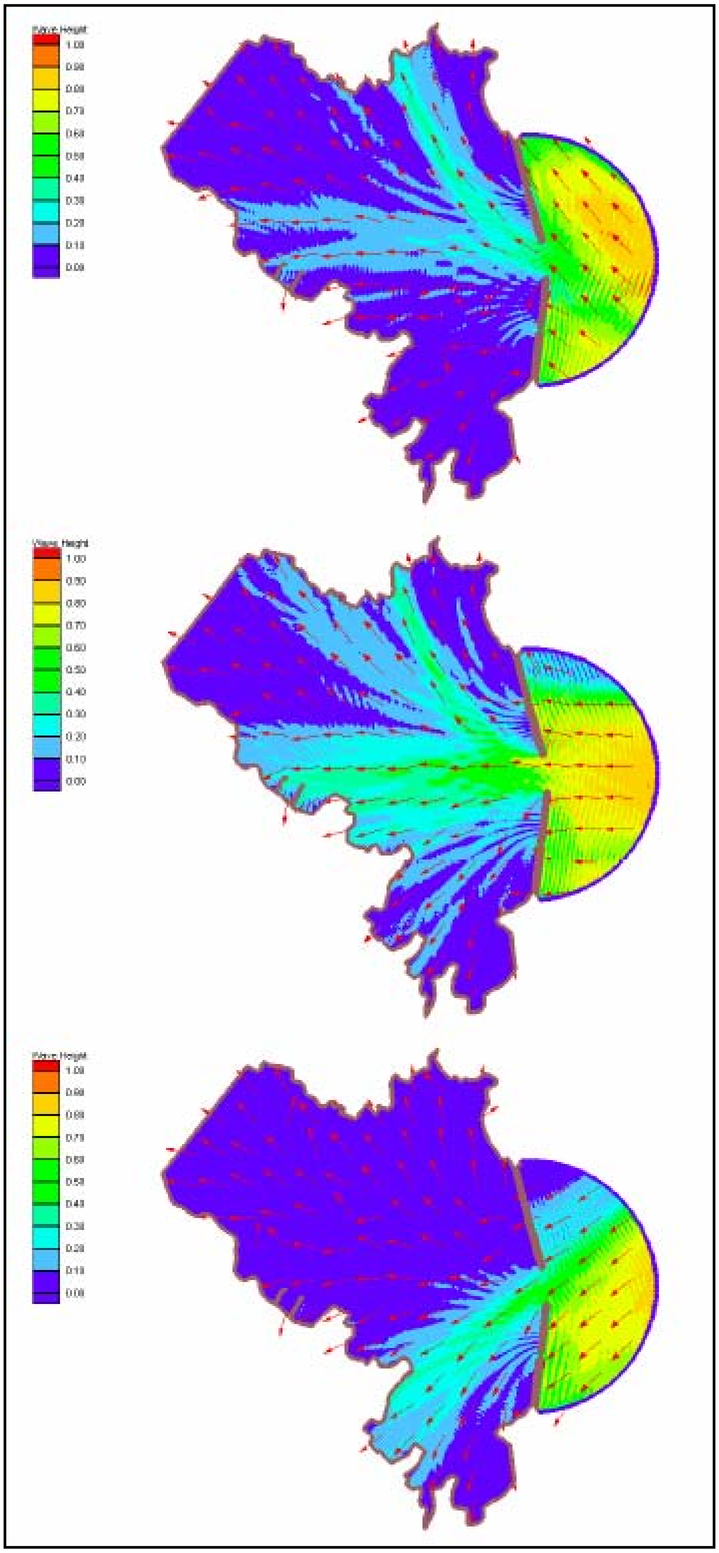


Figure 46. CGWAVE wave height predictions for authorized breakwater configuration with wave conditions of T = 6 sec, H_i = 1 m (3.3 ft), water level = 1 m (3.3 ft) mllw, and wave directions (a) θ = 135 deg (NW), (b) θ = 180 deg (W), and (c) θ = 225 deg (SW).

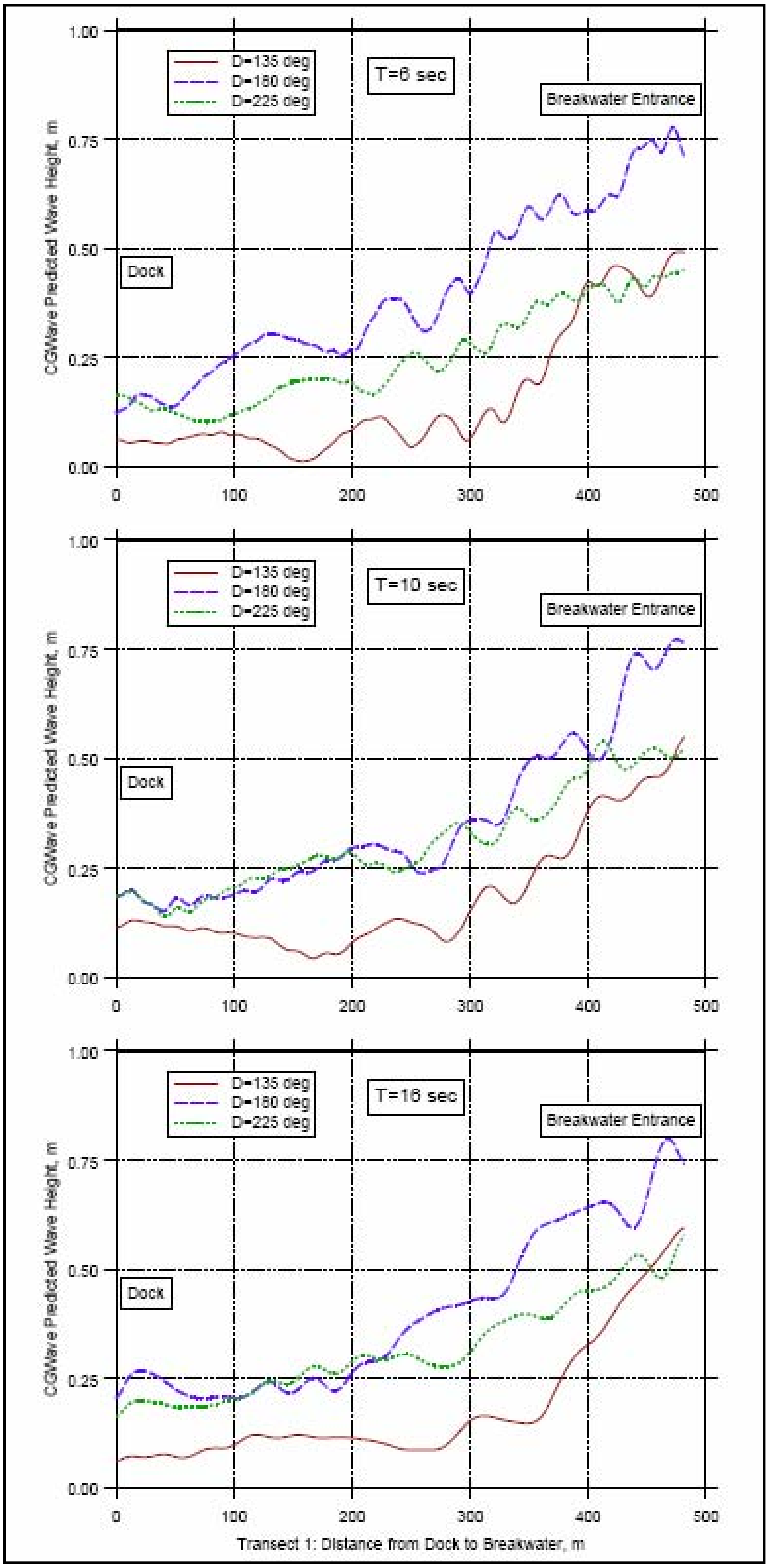


Figure 47. CGWAVE wave height predictions along transect T1 for authorized breakwater configuration with wave conditions of H_i = 1 m (3.3 ft), water level = 1 m (3.3 ft) mllw, and wave periods T = 6 sec, T = 10 sec, and T = 16 sec.

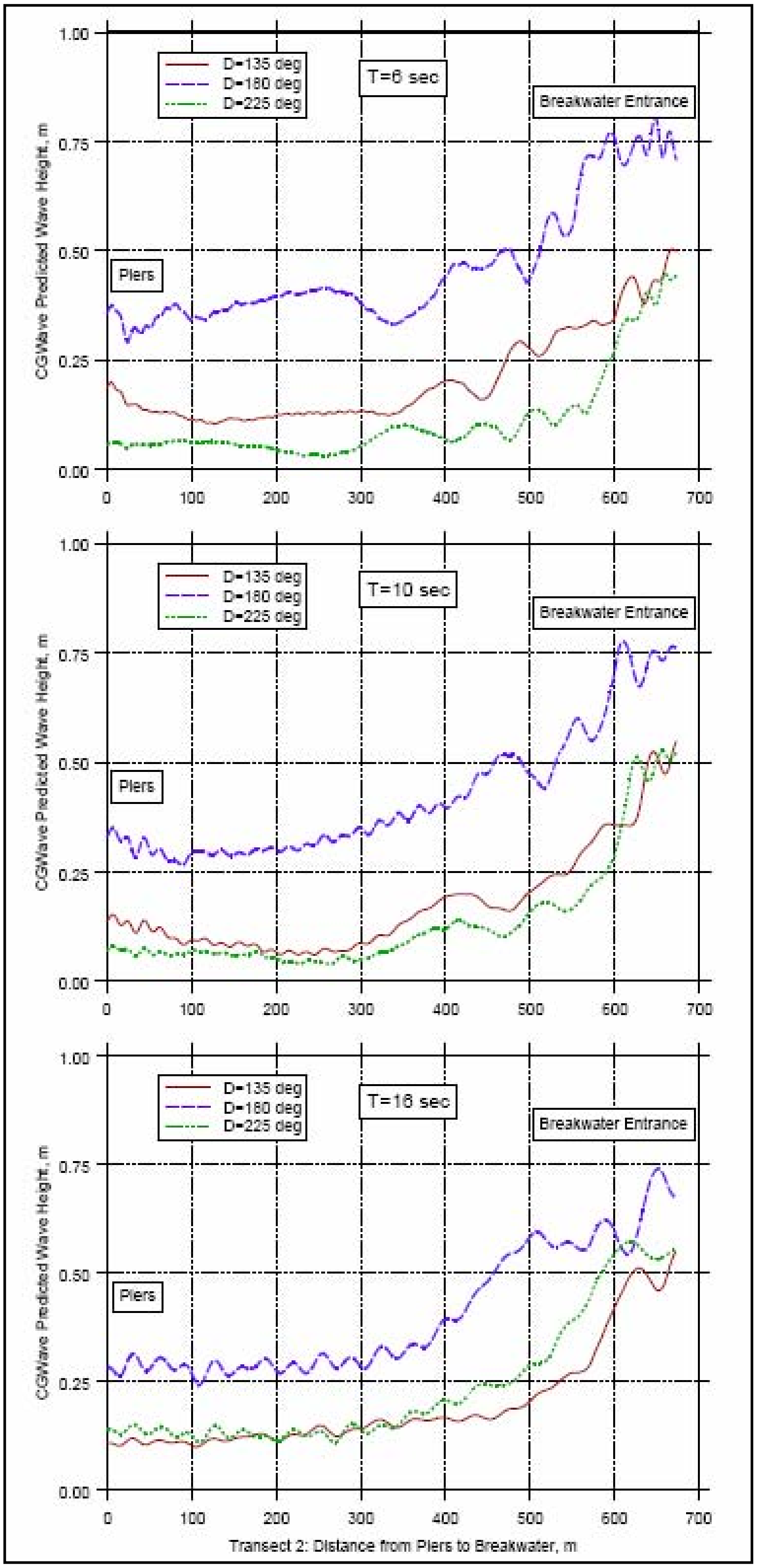


Figure 48. CGWAVE wave height predictions along transect T2 for authorized breakwater configuration with wave conditions of H_i = 1 m (3.3 ft), water level = 1 m (3.3 ft) mllw, and wave periods T = 6 sec, T = 10 sec, and T = 16 sec.

As before, a more meaningful analysis is obtained by looking at the boxes near the high use areas in the vicinity of the public pier and county boat dock. Figure 49 shows the average wave height H for the authorized breakwater configuration for the three wave periods. The maximum wave heights occur for waves traveling to the west for both the public pier and the county boat dock computational boxes. This represents a change from the existing as-built configuration for the county boat dock computational boxes where the maximum heights occurred for a wave direction to the southwest.

Again, the maximum wave heights H_{Max} and their locations from Figure 49 are shown for the public pier and county boat dock computational boxes in Table 10. Averaged over all public pier boxes, the maximum average wave heights H_{All} = 0.24, 0.25, and 0.20 m (0.79, 0.82, and 0.66 ft) for T = 6, 10, and 16 sec, respectively. All maximum values are for waves traveling to the west. Averaged over all county boat dock computational boxes, the maximum average wave heights H_{All} = 0.19, 0.18, and 0.20 m (0.62, 0.59, and 0.66 ft) for T = 6, 10, and 16 sec, respectively. All maximum values are for waves traveling to the west. Finally, the 95-percent confidence interval for average wave height inside all boxes for all wave conditions is 0.13 ± 0.02 m (0.43 ± 0.07 ft) in both the public pier and county boat dock areas.

Comparison of breakwater configurations

In the previous plots, the CGWAVE-predicted wave heights H were shown for each transect, box, wave condition, and breakwater configuration. A comparison of the predicted wave heights between the existing as-built 122-m (400-ft) breakwater gap and the authorized 91-m (300-ft) breakwater gap was developed. Because of the variability along the two transects, only the wave heights in the computational boxes are compared.

Wave height differences H_{Δ} between the existing as-built breakwater configuration and the authorized breakwater configuration are shown for the public pier and the county boat dock computational boxes in Figure 50. Negative differences indicate that the existing as-built configuration has a smaller predicted wave height than the authorized configuration. Therefore, only the positive differences are of interest to the local residents of Tedious Creek since these positive differences imply that the authorized configuration would have reduced the wave energy in the harbor.

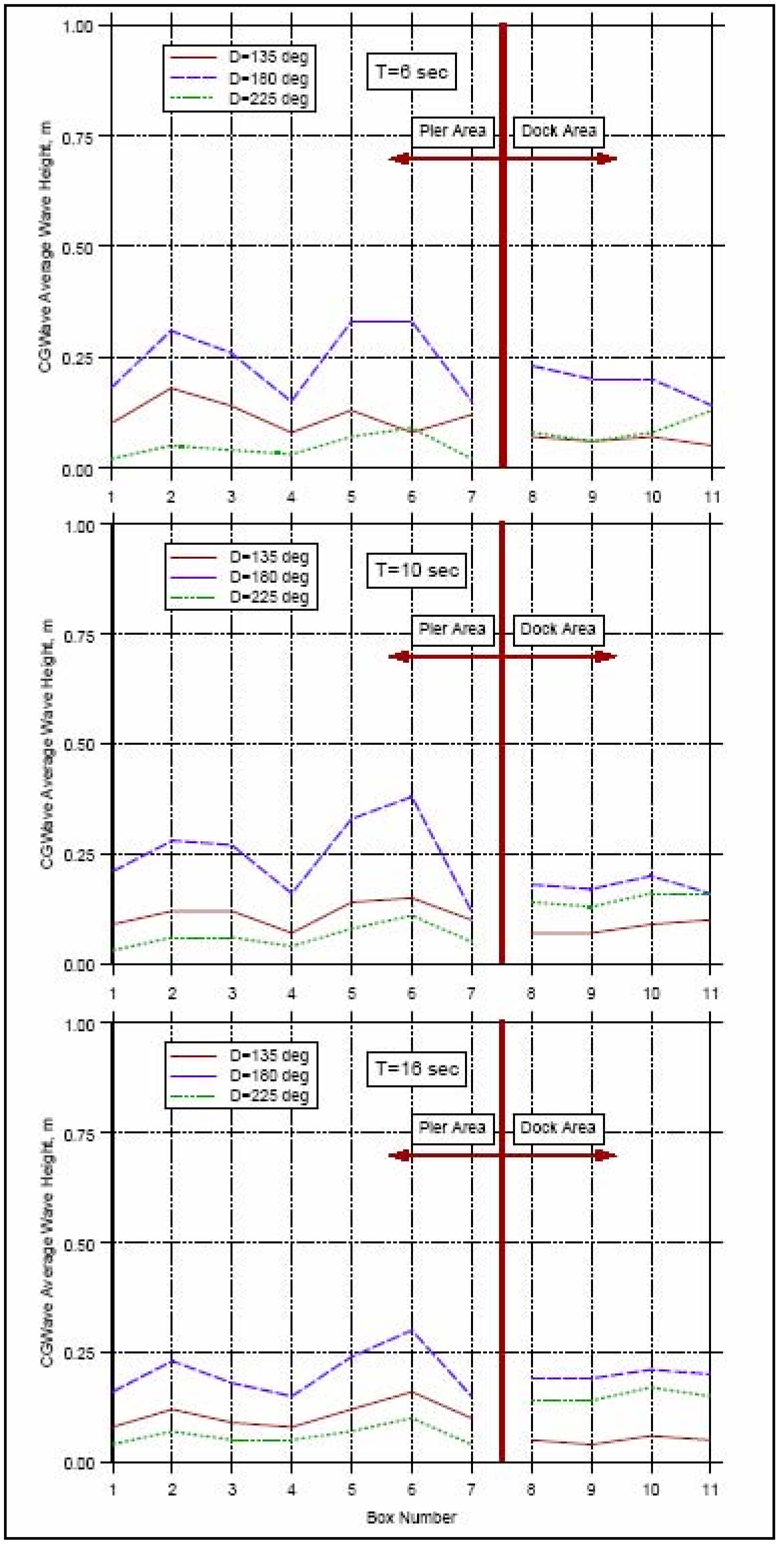


Figure 49. CGWAVE average wave heights in computational boxes for authorized breakwater configuration for waves with H_i = 1 m (3.3 ft) mllw, water level of 1 m (3.3 ft), and wave periods T = 6 sec, T = 10 sec, and T = 16 sec.

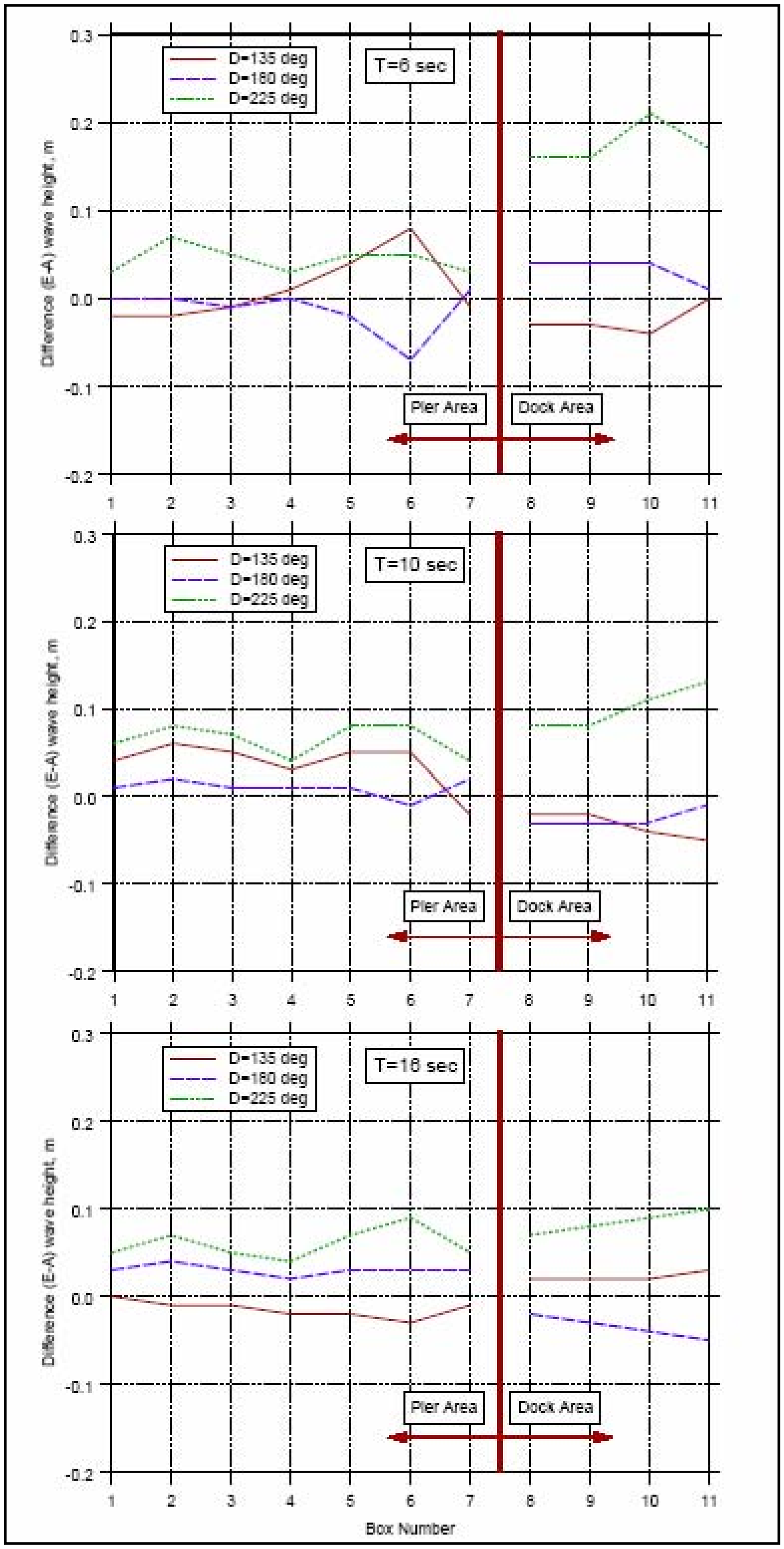


Figure 50. Wave height differences in computational boxes between existing (E) as-built and authorized (A) breakwater configurations for waves with H_i = 1 m (3.3 ft), water level = 1 m (3.3 ft) mllw, and wave periods T = 6 sec, T = 10 sec, and T = 16 sec.

In general, the maximum difference wave heights $H_{\Delta Max}$ are not very large and occurred for southwest waves for both the public pier and the county boat dock computational area boxes. For the public pier boxes, $H_{\Delta Max}$ = 0.08 m (0.26 ft) in box 6 for T = 6 sec; $H_{\Delta Max}$ = 0.08 m (0.26 ft) in boxes 2, 5, and 6 for T = 10 sec; and $H_{\Delta Max}$ = 0.09 m (0.30 ft) in box 6 for T = 16 sec. Averaged over all boxes, the maximum difference wave heights H_{Δ} = 0.04, 0.06, and 0.06 m (0.13, 0.20, and 0.20 ft) for T = 6, 10, and 16 sec, respectively. For the public pier boxes, all values are for waves traveling to the southwest except for T = 6 sec (NW). For the county boat dock computational boxes, the $H_{\Delta Max}$ = 0.21 m (0.69 ft) in box 10 for T = 6 sec; $H_{\Delta Max}$ = 0.13 m (0.43 ft) in box 11 for T = 10 sec; and $H_{\Delta Max}$ = 0.10 m (0.33 ft) in box 11 for T = 16 sec. Averaged over all boxes, the maximum difference wave heights H_{Δ} = 0.18, 0.10, and 0.08 m (0.59, 0.33, and 0.26 ft) for T = 6, 10, and 16 sec, respectively. All values are for waves traveling to the southwest for the county boat dock computational boxes. Finally, the 95-percent confidence interval for average difference wave height inside all boxes for all wave conditions is H_{Δ} = 0.03 ± 0.01 m (0.10 ± 0.03 ft). In summary, the wave height reduction that would have been afforded by the smaller entrance gap had the authorized configuration been constructed is insignificant.

Comparison to previous studies

Offshore and Coastal Technologies performed a numerical model study of Tedious Creek, MD, for the Baltimore District in 2001. They used the Corps' Steady Wave (STWAVE) shallow-water directional spectral model to investigate wave conditions in the vicinity of the county boat dock for (a) pre-project (no-breakwater), (b) existing as-built breakwater project with a 122-m (400-ft) breakwater gap, and (c) authorized breakwater project with a 91-m (300-ft) breakwater gap. The STWAVE finite difference model had a grid spacing of 15.2 m (50 ft), which is much larger than the CGWAVE spacing. The STWAVE model is appropriate for open coast and deepwater applications, but may not be suitable for shallow depths where diffractions, reflection, and nonlinear dispersion could be important.

A JONSWAP spectrum with 30 frequencies and a 3.3 peak enhancement factor and Cosine-4th directional spreading with thirty-five 5–deg direction bins was used. They looked at waves from nine directions from southwest to southeast, three wave heights from 0.5 to 2 m (1.6 to 6.6 ft), and five water levels from 0 to 1.8 m (0 to 6 ft). Not all combinations were

run. Most of the runs were for waves traveling to the southwest, wave height of 1 m (3.3 ft), and water level of 0.73 m (2.4 ft). The extreme storm case had a wave direction of southwest, height of 2 m (6.6 ft), and water level of 1.8 m (6 ft).

As a base case, Offshore and Coastal Technologies ran a “no-breakwater” case of the original harbor before the breakwater was constructed. They found that the existing jetties reduce the incoming wave heights by as much as 70 percent versus a no-breakwater condition. For the location near the county boat dock that Offshore and Coastal Technologies selected, this is equivalent to a 0.3-m (1.0-ft) wave height from an incident 1-m (3.3-ft) wave offshore of the breakwaters. This is comparable to the results obtained with CGWAVE for the existing gap. A “no-breakwater” case was not run using CGWAVE since the breakwater exists, and this was not an alternative that could be considered.

The Offshore and Coastal Technologies Incorporated (2001) study found that wave heights were increased as much as 50 percent at high tide, but negligibly at low tide. This is why CHL selected only the worst-case high-tide level in this study. Wave breaking was the important wave process that contributed to this result for typical daily wave conditions of 0.30 to 0.61 m (1 to 2 ft).

The Offshore and Coastal Technologies Incorporated (2001) study also found that waves traveling to the southwest were the worst-case conditions for waves in the vicinity of the county boat dock. CHL also found this to be true due to the geometry of the harbor and wave refraction.

Finally, Offshore and Coastal Technologies believes that some of the locals’ concerns about larger waves may come from locally generated waves in the creek that travels to the south. This is a condition that may occur daily and can produce waves as large as 0.61 m (2 ft). Unfortunately, the breakwaters at the harbor entrance are not designed for waves from this direction, so no amount of gap closure would alleviate this wave condition. Because these waves develop by the wind blowing over a suitable fetch length, the only type of protection that could be afforded would require some type of jetty(s) along the northern side of the entrance channel inside the harbor or a detached breakwater inside the harbor north of the public pier and/or county boat dock.

Summary

This study provided results from the CGWAVE numerical model study for predicting wave heights in Tedious Creek Harbor, MD. A new breakwater was constructed with an existing as-built gap of 122 m (400 ft) between the breakwater sections on either side of the entrance. An authorized gap of 91 m (300 ft) was originally proposed, but not constructed due to geological and construction concerns. Wave height predictions from the CGWAVE model were compared between the two configurations.

A series of nine regular wave conditions with wave periods of T = 6, 10, and 16 sec, wave directions of θ = 135 (NW), 180 (W), and 225 (SW) deg, and incident wave height of H = 1 m (3.3 ft) were selected as representative wave conditions in Tedious Creek. Predicted wave heights were compared along two transects, T1 and T2, from the breakwater entrance to the county boat dock and to the public pier and 11 rectangular boxes in the vicinity of the county boat dock (four boxes) and public pier (seven boxes). The two transects give a general overview of wave heights inside the harbor, but exhibit significant variability due to the changes in water depth along each one. Averaging wave height in the smaller boxes in the vicinity of the county boat dock and the public pier provides a reasonable way of quantifying the differences in wave energy in the areas of concern to the local residents.

For the existing configuration, larger wave heights occur to the north for waves traveling toward the northwest, to the west and vicinity of the public pier for waves traveling toward the west, and to the southwest and vicinity of the county boat dock for waves traveling toward the southwest. For both transects, the largest wave height was less than 0.78 m (2.6 ft) and the average wave height was less than 0.31 m (1.0 ft). The maximum wave heights in any box for any wave condition were less than 0.37 m (1.2 ft) and 0.30 m (1.0 ft) for the public pier and county boat dock computational boxes, respectively. Finally, the 95-percent confidence intervals for average wave height inside all boxes for all wave conditions are 0.16 ± 0.02 m (0.52 ± 0.07 ft) in the public pier area and 0.16 ± 0.03 m (0.52 ± 0.10 ft) in the county boat dock area.

For the authorized configuration, the narrower gap reduces waves traveling to the southwest more than to the west along both transects. The difference in wave height for waves traveling to the west and waves traveling to the southwest decreases as wave period increases, however.

The effect of the smaller gap was minimal on overall wave height reduction, in agreement with the Offshore and Coastal Technologies Incorporated (2001) results. The largest wave height along the two transects was 0.86 m (2.8 ft), with an average slightly less than the existing configuration. The difference in maximum wave heights is probably due to reflections at the breakwater entrance from the longer breakwater in the authorized configuration. The maximum wave heights in any of the boxes for any wave conditions in the public pier and the county boat dock areas were 0.38 m (1.3 ft) and 0.23 m (0.8 ft), respectively. Finally, the 95-percent confidence interval for average wave height inside all boxes for all wave conditions was 0.13 ± 0.02 m (0.43 ± 0.07 ft) in both the public pier and the county boat dock areas.

Wave height differences were calculated between the wave heights predicted for the existing 122-m (400-ft) gap and the authorized 91-m (300-ft) gap. Because of the variability along the two transects, only the wave heights in the boxes were compared. In some cases, the existing configuration had lower wave heights. Only the positive differences were reported here as these represent cases where the authorized configuration would have resulted in smaller waves inside the harbor. In general, the maximum difference wave heights were not very large and occurred for southwest waves for both the public pier and the county boat dock boxes. The largest wave height differences were less than 0.09 m (0.3 ft) and 0.21 m (0.7 ft) for the public pier and the county boat dock boxes, respectively. Finally, the 95-percent confidence interval for the average difference wave height inside all boxes for all wave conditions was 0.03 ± 0.01 m (0.10 ± 0.03 ft). In conclusion, any wave height reduction that would have been afforded by the smaller entrance gap had the authorized configuration been constructed is truly insignificant.

Comparisons with the Offshore and Coastal Technologies Incorporated (2001) STWAVE numerical model study are in general agreement with the wave heights and directions predicted by their model.

5 RMA2 Hydrodynamic Verification and RMA4 Flushing Analysis of Tedious Creek Existing As-Built Condition[1]

This chapter addresses the functionality of the circulation and flushing aspects of the Tedious Creek existing as-built condition (122-m (400-ft) breakwater gap opening) by applying two models (RMA2 and RMA4) within the TABS Multi-Dimensional system (TABS-MD) suite of numerical models and utility programs. The TABS-MD system is interfaced within the Department of Defense Surface Water Modeling System (SMS) for graphics and efficient implementation of pre- and post-processing.

The RMA2 model was used to demonstrate general hydrodynamic circulation patterns resulting from verification to the August 2001 field data, and the RMA4 model was used to analyze harbor flushing.

RMA2 – Two-dimensional hydrodynamic model

The heart of the TABS-MD system is RMA2, the hydrodynamic model. RMA2 is a two-dimensional, depth-averaged, free-surface, finite element program for solving hydrodynamic problems. RMA2 was originally developed by Norton et al. (1977), Resource Management Associates, Inc., Davis, CA. Modifications to the original code have been made by a number of CHL researchers.

RMA2 solves the two-dimensional, depth-averaged equations governing shallow water. It uses the Reynolds form of the nonlinear Navier-Stokes equations, and includes phenomenological terms such as bed shear stress, wind stress, wave stress, and coriolis effects. RMA2 is a finite element model for subcritical open-channel flow. It computes velocities, flow rates, and water surface elevations in rivers, estuaries, wetlands, etc. Two of RMA2's more popular features are the automatic parameter selection and the model's ability to handle wetting and drying. A complete user's guide documentation of RMA2 is available for download at *http://chl.erdc.usace.army.mil/chl.aspx?p=s&a=ARTICLES;480*.

[1] This chapter was written by Barbara P. Donnell, Mathematician, Hydrologic Systems Branch, Flood and Storm Protection Division, CHL.

The serial (frontal) version of RMA2 version 4.56 was used for this Windows personal computer application.

RMA2 model using SMS

The RMA2 model is interfaced with the SMS. The SMS is a comprehensive graphical user interface for model conceptualization, mesh generation, statistical interpretation, and visual examination of the hydrodynamic model simulation results. Typically there are two input files for RMA2: (a) a geometry file (*.geo) describing the mesh and bathymetry, and (b) a run control file (*.bc) describing run control parameters and the boundary conditions. As a pre-processor, the SMS interface was used to conceptualize the Tedious Creek domain, generate the finite element geometry, and construct the basic RMA2 run control file for the August 2001 data set.

Finite element mesh geometry

The SMS conceptualization of Tedious Creek was built with reference to the registered aerial image obtained during the monitoring program. The horizontal coordinate system is the State Plane North American Datum (NAD) 27 (US) Maryland 1900, and the vertical reference is the National Geodetic Vertical Datum (NGVD) 29 (US), in feet. For this application, the finite element mesh is comprised of two-dimensional (2D) isoparametric-based triangle and quadrilateral elements. The isoparametric formulation allows the mesh geometry to precisely follow the channel alignment, land features, etc. The 2D mesh, referenced state plane units of feet, was composed of 5,190 elements, 14,192 nodes, and 7 material properties. The finest resolution was placed near the structures and within the gaps. Typical element area sizes in the vicinity of the gaps varied between 3 and 93 sq m (30 and 1,000 sq ft), with element edge dimensions ranging from 0.9 to 18.3 m (3 to 60 ft).

The August 2001 bathymetric survey was imported into SMS as scatter data (x,y,z-values) and interpolated onto mesh nodes. The 2001 bathymetric color contours are presented for the entire computational domain in Figure 51 and then shown overlaid with the elements and zoomed to the primary study area in Figure 52.

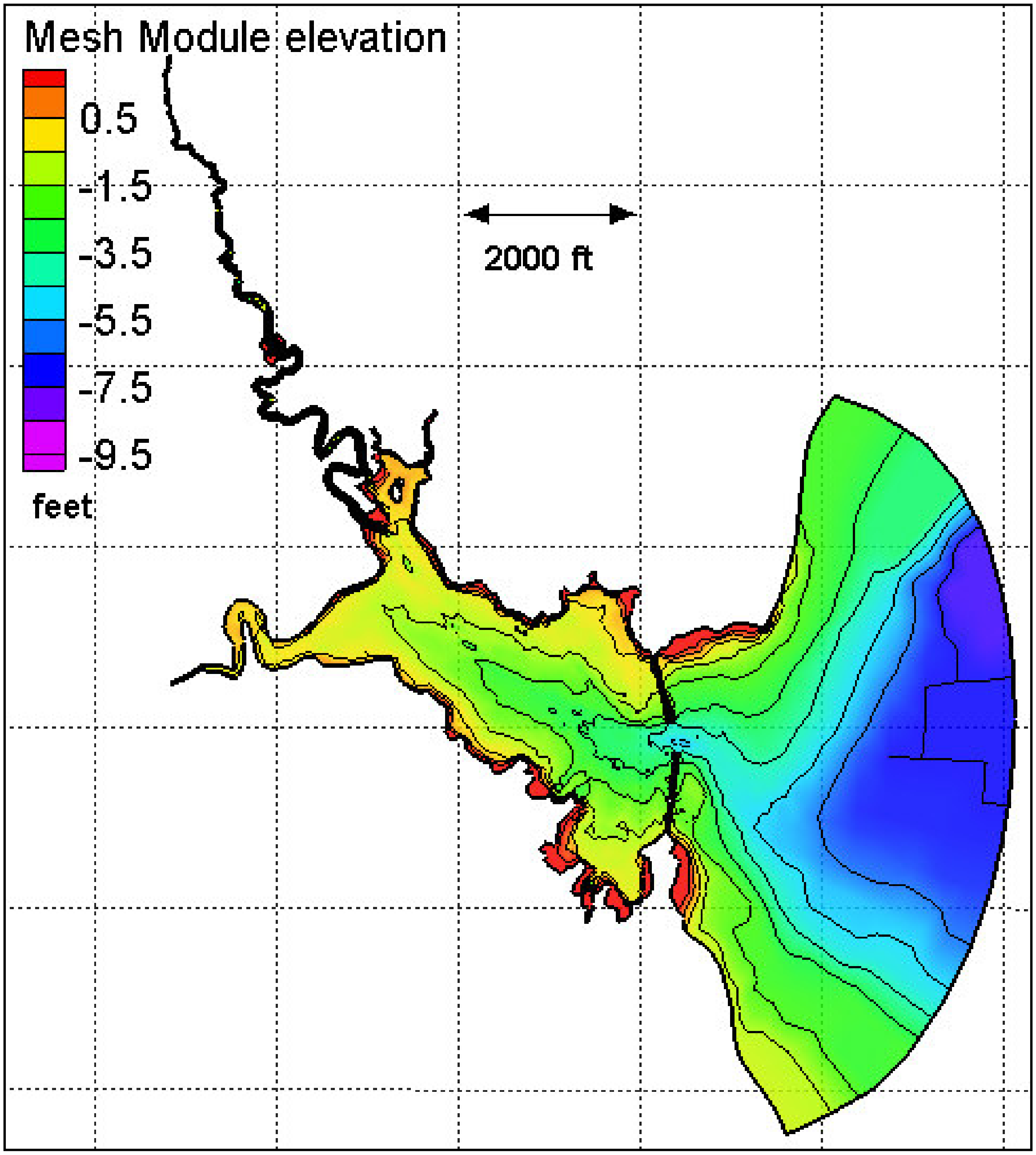
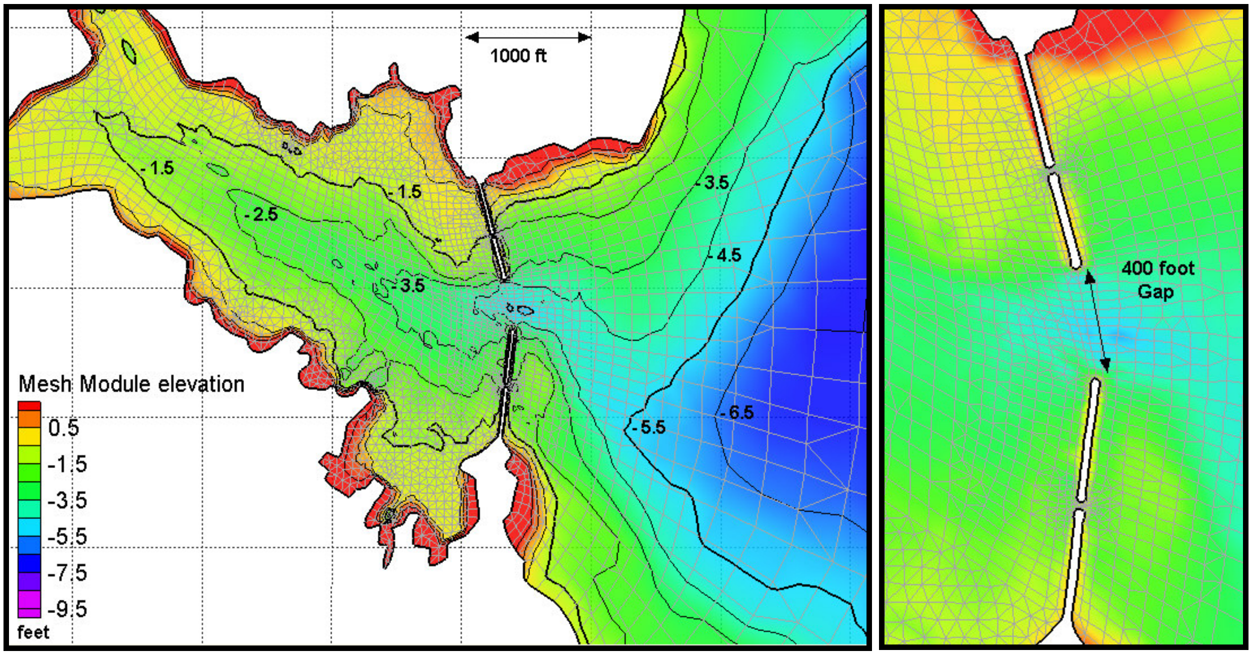


Figure 51. Tedious Creek bathymetric contours of the computational domain, August 2001 survey (elevation ft, NGVD).



a. Primary study area b. Gap details

Figure 52. Tedious Creek bathymetric contours of (a) central harbor study area, and (b) closeup of gap openings, August 2001 survey (elevation ft, NGVD).

The mesh material assignments for each element were grouped into zones. This provided flexibility in assigning the numerical model parameters. Figures 53 and 54 are color-coded to illustrate the material assignments. Table 11 lists the seven material types at the site and provides a summary of the minimum, maximum, and average bottom elevations for each group.

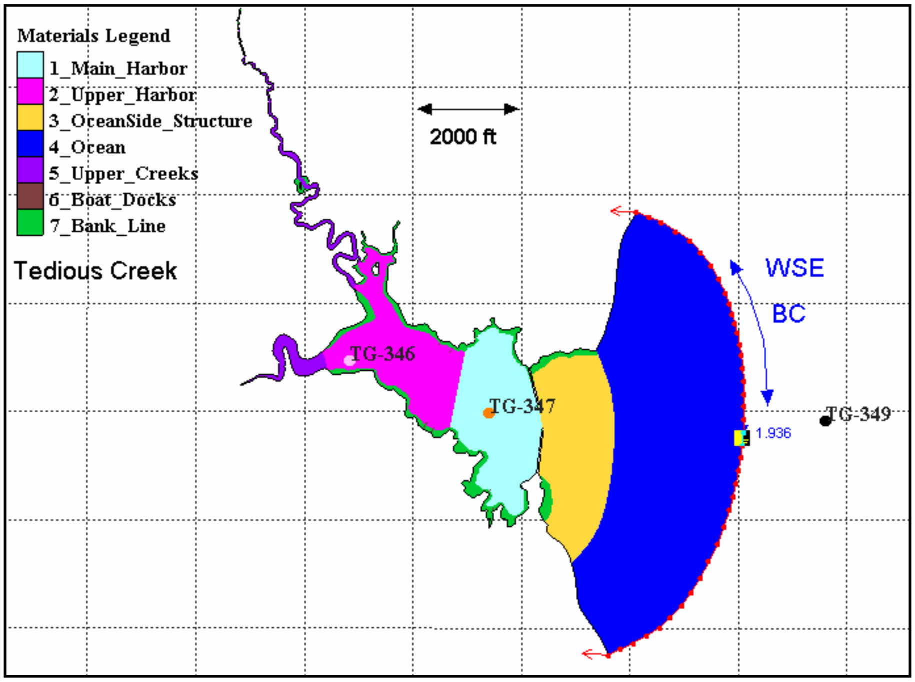


Figure 53. Tedious Creek RMA2 material assignments, field tide gages, and tidal boundary condition location.

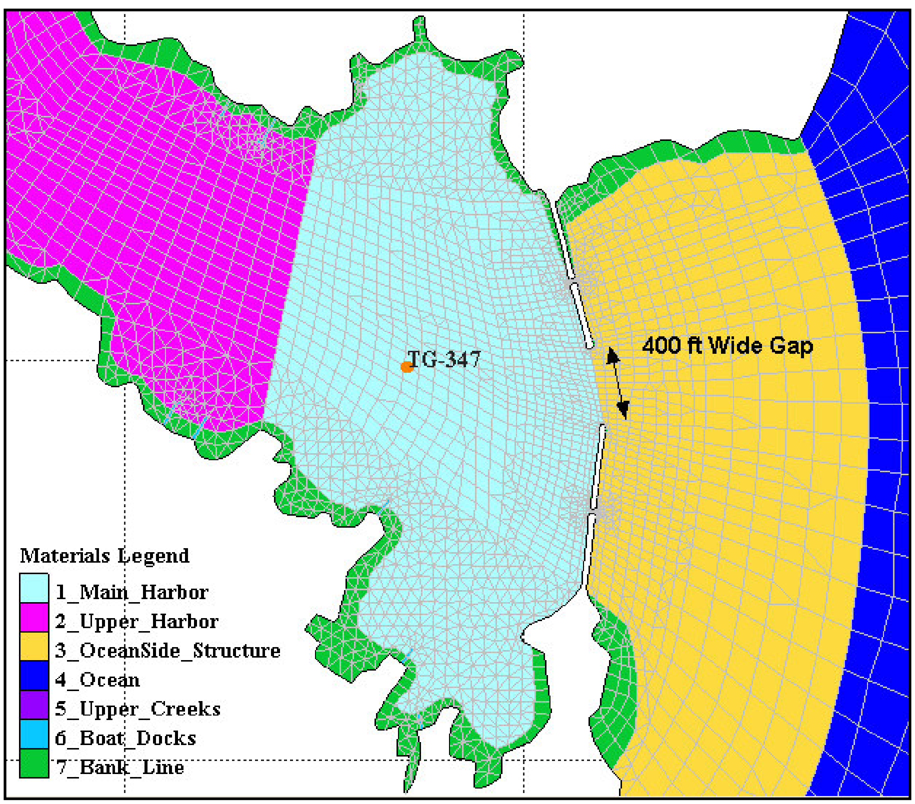


Figure 54. Tedious Creek RMA2 mesh elements and corresponding material assignment for the primary study area.

Table 11. Material type groupings, description, and characteristic bottom elevation.

Material Geometry Bottom Elevation Breakdown by material:				
Type IMAT	Label	Minimum Z-Val (ft)	Maximum Z-Val (ft)	Average Z-Val (ft)
1	Mid Harbor	-5.77	2.63	-1.94
2	Upper Harbor	-3.50	3.50	-0.92
3	Ocean Entrance	-5.97	2.40	-2.73
4	Ocean	-8.00	2.79	-4.80
5	Upper Creeks	-12.07	3.46	-1.09
6	Boat Docks	-3.00	4.00	0.39
7	Bank Line	-3.08	4.71	1.59

RMA2 run control

Once the mesh was completed, SMS was used to assign the run control parameters. First among these parameters was the time and iteration control, followed by the tidal boundary condition, turbulent exchange coefficient, roughness coefficients, wetting and drying, and run time performance.

Timing and iteration control

The timing control for the RMA2 model is assigned on the TZ-Card, shown below. The first parameter on this control card is the time step (0.25 hr), followed by the maximum number of simulation hours (130.5 hr), and the maximum number of time steps (130.5 hr x 4 time steps/hr).

The TI-Card dictates the iteration and convergence criterion. As shown, four iterations per transient time step and a water depth convergence criterion of 0.15 cm (0.005 ft) were specified. If the depth convergence is achieved prior to the fourth iteration, then the model would consider the solution to be sufficiently converged and proceed to the next time step.

```
TZ 0.25 130.5 522 0 0
TI 4 4 0.0010 0.0050
```

Tidal boundary condition

Since the creeks are dead-ended tidal creeks with no freshwater inflows, the only boundary condition for the RMA2 model is the measured tide

gage values recorded during the August 2001 survey. Figure 53 highlights the location of tide gage 349 and shows the semi-circular-shaped tidal boundary condition string (shown in red dots in figure). All nodes along this line (i.e., node string) are identically assigned the same water surface elevation for each transient time step comprising the simulation. This is accomplished by use of the BHL-Card for each time step. A representative sample of the BHL-Cards for the first few time steps is shown below.

```
BHL 1  1.936
END Simulation at time = 0.00
BHL 1  2.051
END Simulation at time = 0.25
BHL 1  2.144
END Simulation at time = 0.50
BHL 1  2.282
END Simulation at time = 0.75
BHL 1  2.328
END Simulation at time = 1.00
etc.
```

The time step increment for the simulation is 0.25 hr. Figure 55 shows tide gage 349 field values that were assigned to the water surface elevation boundary condition for the 130.5-hr RMA2 simulation. The minimum and maximum water surface elevation values are 0.23 and 3.62 ft, respectively.

Turbulent exchange coefficient

Since RMA2 uses the Galerkin method of weighted residuals, there is no inherent artificial diffusion. Therefore, a certain amount of added turbulence is required to achieve stability. Turbulence may be defined generally as the effect of temporal variations in velocity and the momentum exchange associated with their spatial gradients. In particular, turbulence is viewed as the temporal effects occurring at time scales smaller than the model time step. For this application, the automatic method known as the Peclet formulation was used to adjust the turbulent exchange coefficient (i.e., E, the eddy viscosity) for each element after completion of each iteration of the simulation. The Peclet method is activated by the PE-Card within the RMA2 run control file. The Peclet number defines the relationship between the average elemental velocity magnitude, elemental length, fluid density, and turbulent exchange coefficient. The Peclet equation is presented as:

$$P = \frac{\rho VL}{E} \tag{1}$$

where

P =Peclet number
ρ = fluid density (1.00 g/cu cm (1.94 slugs/cu ft)
V =velocity (ft/sec)
L =length of element (ft)
E =eddy viscosity (lb-sec/sq ft)

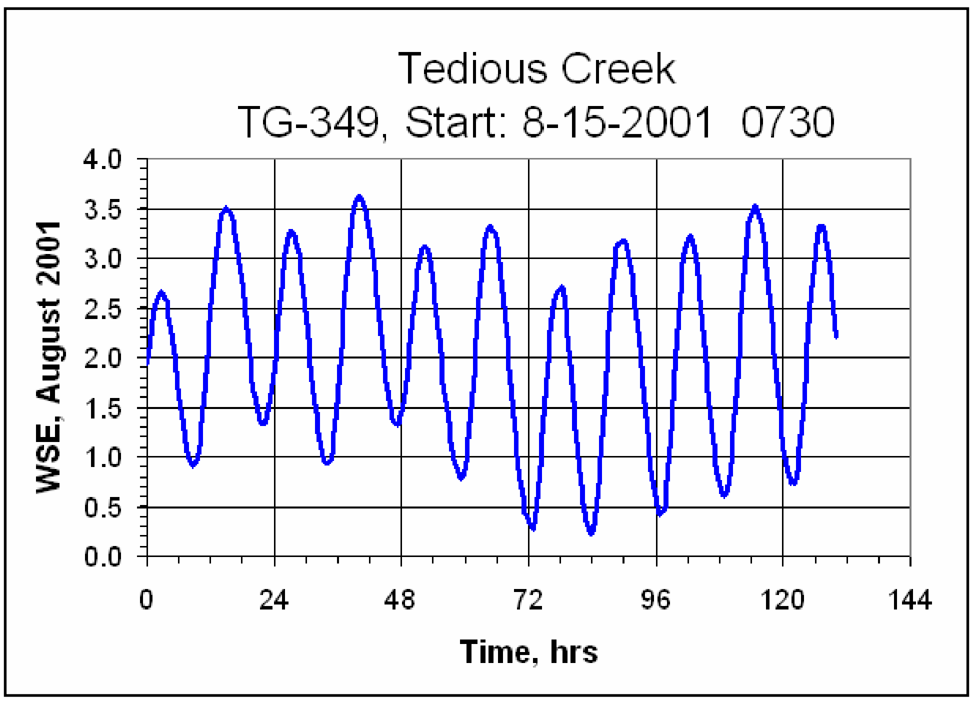


Figure 55. Tidal water surface elevation (WSE) boundary condition (ft, NGVD).

For the Tedious Creek application, a global Peclet number of 10 was used. If the computed velocity magnitude within an element was less than the 30.48 cm/sec (1.0 ft/sec), then 30.48 cm/sec (1.0 ft/sec) was used in the Peclet equation. Scaling factors were all 1.0 for the xx,xy,yx,yy-directional components of the eddy viscosity.

```
PE 1 10 1.0 1. 1. 1. 1.
```

Roughness coefficients

The next items required within the RMA2 run control are the bottom roughness coefficients (dimensionless parameter). This application used the automatic assignment of Manning's n-value based upon transient water depth, which is governed by the RD-Card as shown below.

```
RD   1  2  0.02  3  0.025  0.08       .02  0.05
RDT  7  2  0.04  2  0.030  0.166667 .02  0.10
```

Figure 56 illustrates the two curves used to govern the automatic Manning's n-value assignments. The global curve was used throughout the computational domain, except for the material type 7, which comprises a narrow strip of elements along the shallow bank line. The global assignment (blue curve) dictates that as the water depth falls during low tide, the roughness would increase but is limited to a range between 0.02 and 0.05. For the bank line (green curve) that can become very shallow during low tide, the roughness exponentially climbs but is limited to a range between 0.02 and 0.10.

Wetting and drying

Since one emphasis of the Tedious Creek hydrodynamic modeling is performance in the vicinity of the structures, the bathymetric elevations around the structures honored the "water's edge" field measurements. Given the tidal environment, this meant that there would be times during the simulation when the structures' edge nodes would be dry. The Marsh Porosity transitional wet/dry technique within RMA2 is well suited for this situation. By using the DA-Card, this technique can be automated such that the marsh porosity parameters are adjusted to allow for each node in the domain to remain "transitionally" wet. The parameter settings for the DA-Card used in the Tedious Creek verification are shown below:

```
DA  1  2  0.60  0.02  -4.0
```

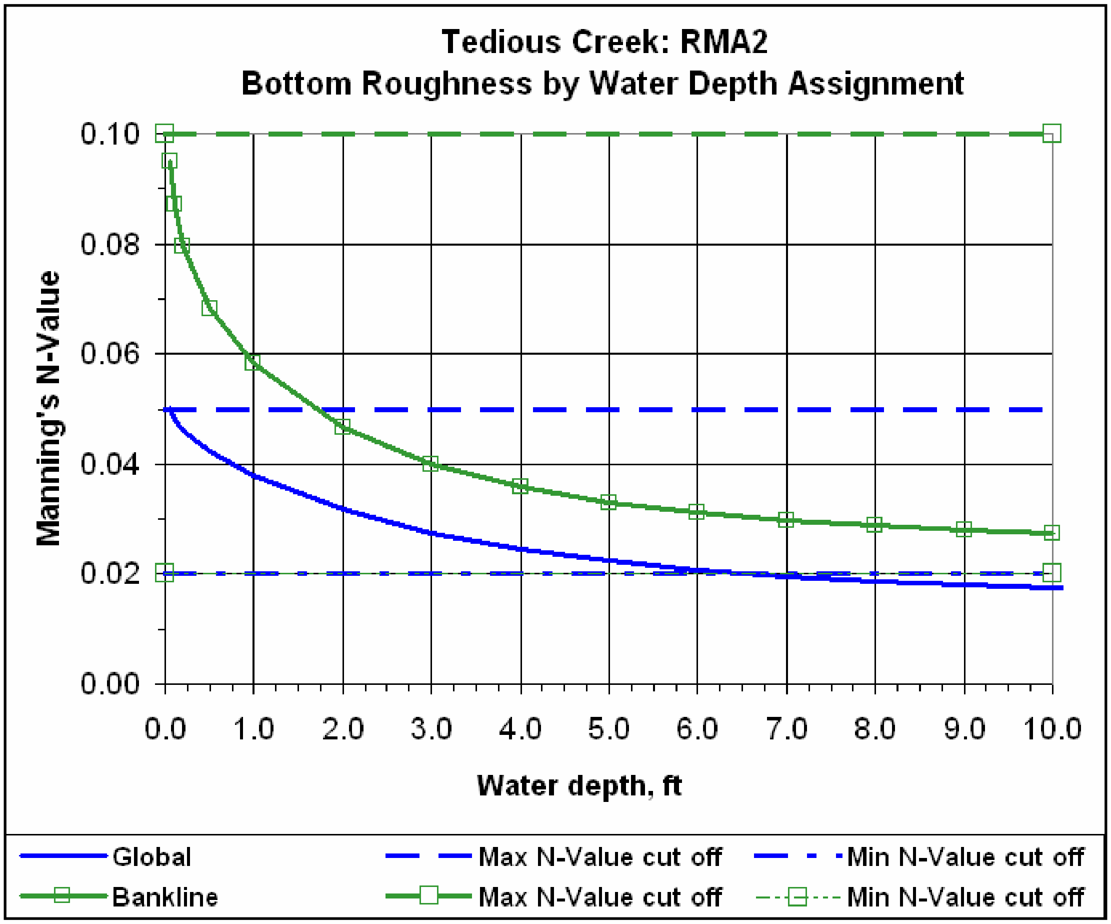
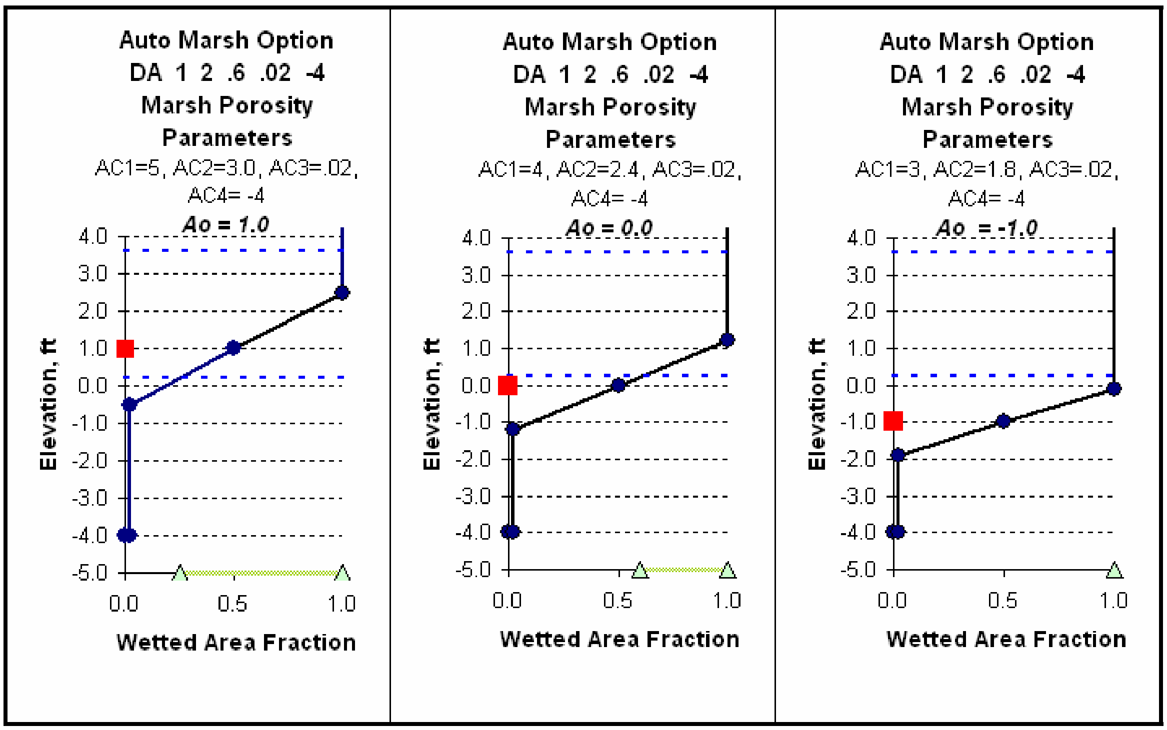


Figure 56. RMA2 automatic Manning's n-value bottom roughness assignment based on water depth.

Figure 57 provides three example marsh porosity wetted surface area transition plots for three bottom elevations of +0.3 m (+1.0 ft), 0.0 m (0.0 ft), and -0.3 m (-1.0 ft) NGVD. The title of each example shows the marsh porosity parameters that were automatically computed based upon the DA-Card assignment. The red square symbol marks the bottom elevation where the wetted surface area factor is 0.5 by definition. The blue dashed line marks the minimum and maximum height of the tidal water surface elevation applied as a boundary condition. The black solid "funnel shaped" curve represents the shape of the wetted surface area fraction as a function of the tidal water surface elevation. The active wetted area fraction that results for each example is marked with a green line and triangle symbols. For this automatic marsh porosity simulation, all elements in the computational domain will remain "functionally wet" with 1.3 m (4.23 ft) of water depth multiplied by a percentage of the wetted capacity of the element that is filled with water during the lowest elevation of the tidal boundary condition (0.07 m (0.23 ft)).

Figure 57 illustrates that, as the bottom elevation increases, the transition range (AC2) becomes larger, which results in a more gradual wet/dry transition. As shown in the left-most examples, the 0.3-m (1.0-ft) bottom elevation has the most gradual wet/dry transition. For this case, a transitional wetting and drying will occur as the tidal boundary condition fluctuates between 0.07 m and 1.1 m (0.23 and 3.62 ft) because the wetted area fraction varies between 0.25 and 1.0. In contrast, as shown in the right illustration, for locations with a -0.3-m (-1.0-ft) bottom elevation, the wetted surface area fraction will always equal 1.0 (fully wet) for all elevations of the tidal boundary condition.



a. Large transition range b. Medium transition range c. Small transition range

Figure 57. Example automatic marsh porosity diagrams for nodes with bottom elevations of +0.3 m (+1.0 ft), 0.0 m (0.0 ft), and -0.3 m (-1.0 ft).

Run time performance

The SMS interface allows the modeler to select the best mesh renumbering sequence to achieve optimal computational performance. The finite element mesh renumbering resulted in a computational “maximum front width” of 409.

The dynamic memory version of RMA2 permits the user to customize the dimensions of the simulation. This is accomplished by providing RMA2 with a file named "r2memsize.dat" as shown below. Of particular importance is the buffer size (NBS) setting. To optimize run time performance, the NBS parameter must be of sufficient size to allow the RMA2 matrix inversion to fit within memory rather than writing "buffer blocks" to disk files. A NBS value of 12 million was required to have no buffer writes and to fit the Tedious Creek application into memory.

```
r2memsize.dat
user
MND    MEL   MR1   MFW   NBS       MPB   MCC   MCCN  MXSTRM  MXUU  MWTS
15000  6000  45000  500  12000000  50000  150   300    1       100   10
```

The RMA2 diagnostic file "r2sol.dat," which is written during the simulation, summarizes the number of iterations to achieve the specified convergence for each time step. A summary of these data is presented below. The RMA2 simulation averaged 2.6 iterations per time step and never used less than 2 iterations per time step. For the 130.5-hr simulation, the average convergence was 0.0005 m (0.00165 ft). The maximum depth convergence of 0.0033 m (0.01082 ft) occurred at simulation hr = 62.5, near the occurrence of maximum flood currents.

```
r2sol.dat
Time-Step                  Time    UVH_ Iter Depth_Conv
UVH_Sol Record = 1.        0.000  2      0.00010
UVH_Sol Record = 2.        0.250  4      0.00024
UVH_Sol Record = 3.        0.500  3      0.00177
UVH_Sol Record = 4.        0.750  3      0.00135
UVH_Sol Record = 5.        1.000  3      0.00058
. . .Skip
UVH_Sol Record = 521.  130.000  2      0.00153
UVH_Sol Record = 522.  130.250  3      0.00215
UVH_Sol Record = 523.  130.500  4      0.00052

AVERAGE NUMBER OF ITERATIONS/TIME STEP = 2.6
MINIMUM and MAXIMUM DEPTH CONVERGENCE = 0.00005 / 0.01082 ft
AVERAGE DEPTH CONVERGENCE/TIME STEP = 0.00165 ft
```

The Tedious Creek RMA2 (serial/frontal version) simulation was run on a Dell Dimension 8200, 2.8-MHz Windows XP personal computer with

2 gigabytes of memory. The total simulation time for 130.5 hr of simulation was 3.5 CPU hr.

RMA2 verification to August 2001 event

The RMA2 model simulation results were compared to available field data for the August 2001 survey. These comparisons demonstrate that the RMA2 model for Tedious Creek as-built existing conditions is verified.

Statistical error measures

One of the difficulties in comparing model and field data is the lack of appropriate measures with which to make the judgments of the comparisons. Standard statistical measures such as the mean, variance, and skewness are not particularly helpful. The SMS graphical user interface uses the mean absolute error and root mean square error as indicators. These, and other statistical error measures, were used to compare scalar field data and numerical model calculations. These error measures are presented below.

Mean absolute error (MAE). As shown in Equation 2 below, the MAE is the average of the absolute value of the computed minus the observed (the error). For this measure, the goal is "the smaller the MAE, the better." The units will be the same as the scalar values being compared.

$$MAE = \frac{\sum_{i=1}^{n} \left| C_i - O_i \right|}{n} \tag{2}$$

Root mean square error (RMSE). As shown in Equation 3 below, the RMSE is the square root of the mean squared error. Again, for this measure, the goal is "the smaller the RMSE, the better." The units will be the same as the scalar values being compared.

$$RMSE = \sqrt{\frac{\sum_{i=1}^{n} \left(C_i - O_i \right)^2}{n}} \tag{3}$$

Coefficient of efficiency (COE). For the COE measure, the best fit is to acquire a value as close to 1.0 as possible from Equation 4.

$$E = 1 - \frac{\sum_{i=1}^{n} (O_i - C_i)^2}{\sum_{i=1}^{n} (O_i - \overline{O})^2} \qquad (4)$$

Coefficient of determination (COD). This statistic is widely used in regression analysis. For the COD measure, Equation 5, the best indicator is a value close to 1.0.

$$R^2 = \left[\frac{\sum_{i=1}^{n} (C_i - \overline{C})(O_i - \overline{O})}{\sqrt{\sum_{i=1}^{n} (C_i - \overline{C})^2} \sqrt{\sum_{i=1}^{n} (O_i - \overline{O})^2}} \right]^2 \qquad (5)$$

where

n =number of comparisons between the computed and observed values
C_i =computed value of the i-th comparison
O_i =observed value of the i-th comparison
$\overline{C}$ =mean of the computed over the n comparisons
$\overline{O}$ =mean of the observed over the n comparisons

Water surface elevation verification

There were three field tide gages that recorded water surface elevation for the 130.5 hr of the August 2001 survey period. The data from the three gage locations shown in Figure 53 are plotted together and presented in Figure 58. The signals appear to have no phase shift and only a slight difference in amplitude. The mean value plotted for the boundary gage TG-349 is 0.62 m (2.04 ft).

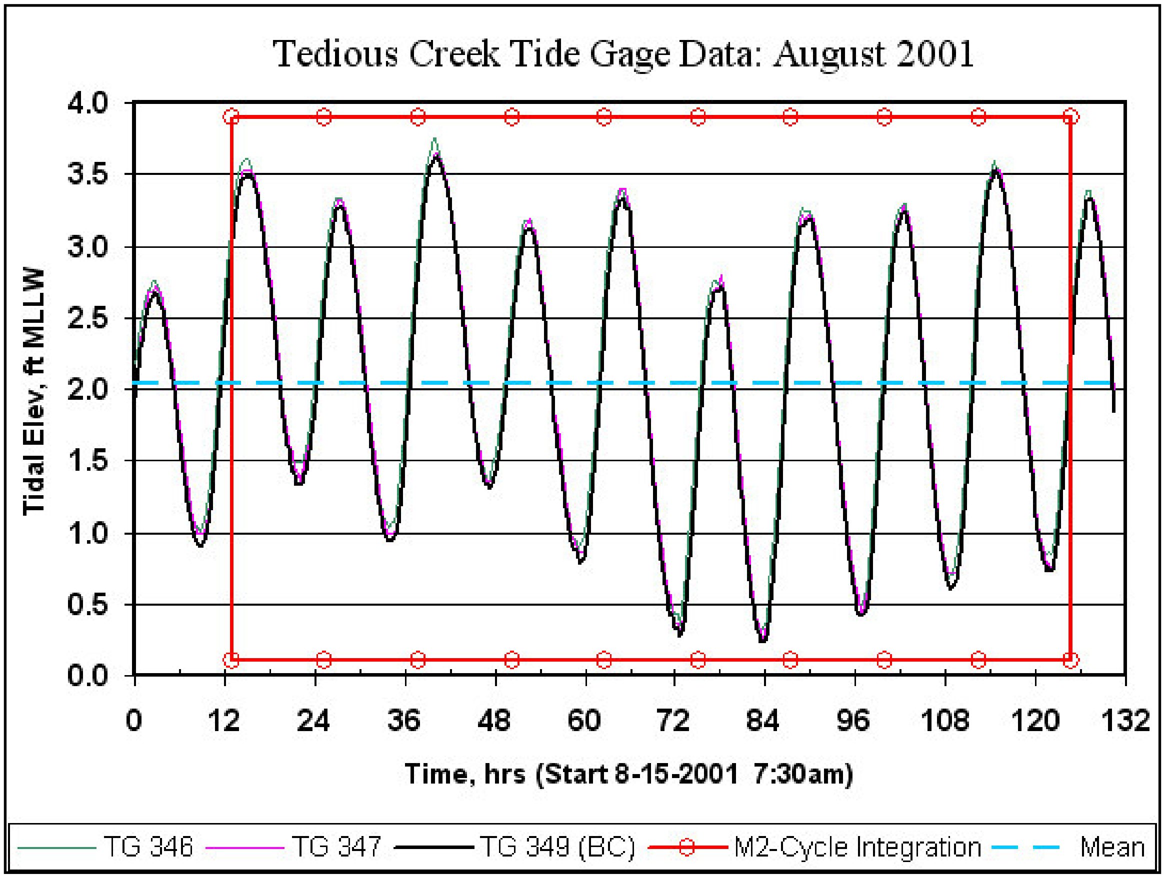


Figure 58. Three field tide gages plotted for August 2001 survey period.

The two field tide gages (TG-346 and TG-347) that fall within the numerical model domain were compared to the solution obtained by the RMA2 numerical model. There is excellent agreement in the comparisons, which is demonstrated both graphically and statistically.

The graphical comparisons of model versus measured water surface elevations are provided in Figures 59 and 60. These figures show the full 130.5-hr simulation and a 2-day enlargement of the model versus field data for tide gages 346 and 347.

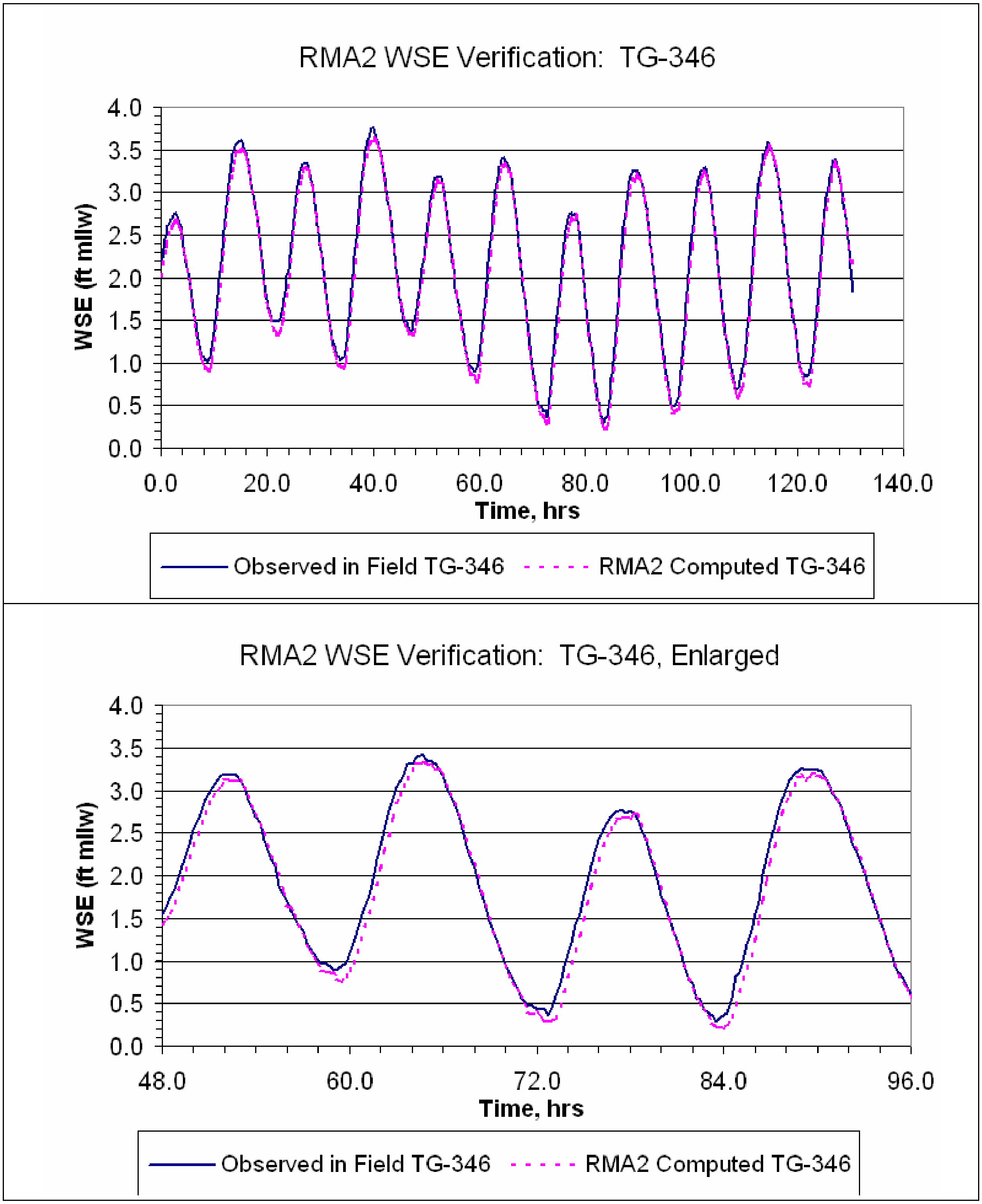


Figure 59. Water surface elevation (WSE) measurement at field tide gage 346 versus RMA2 computations.

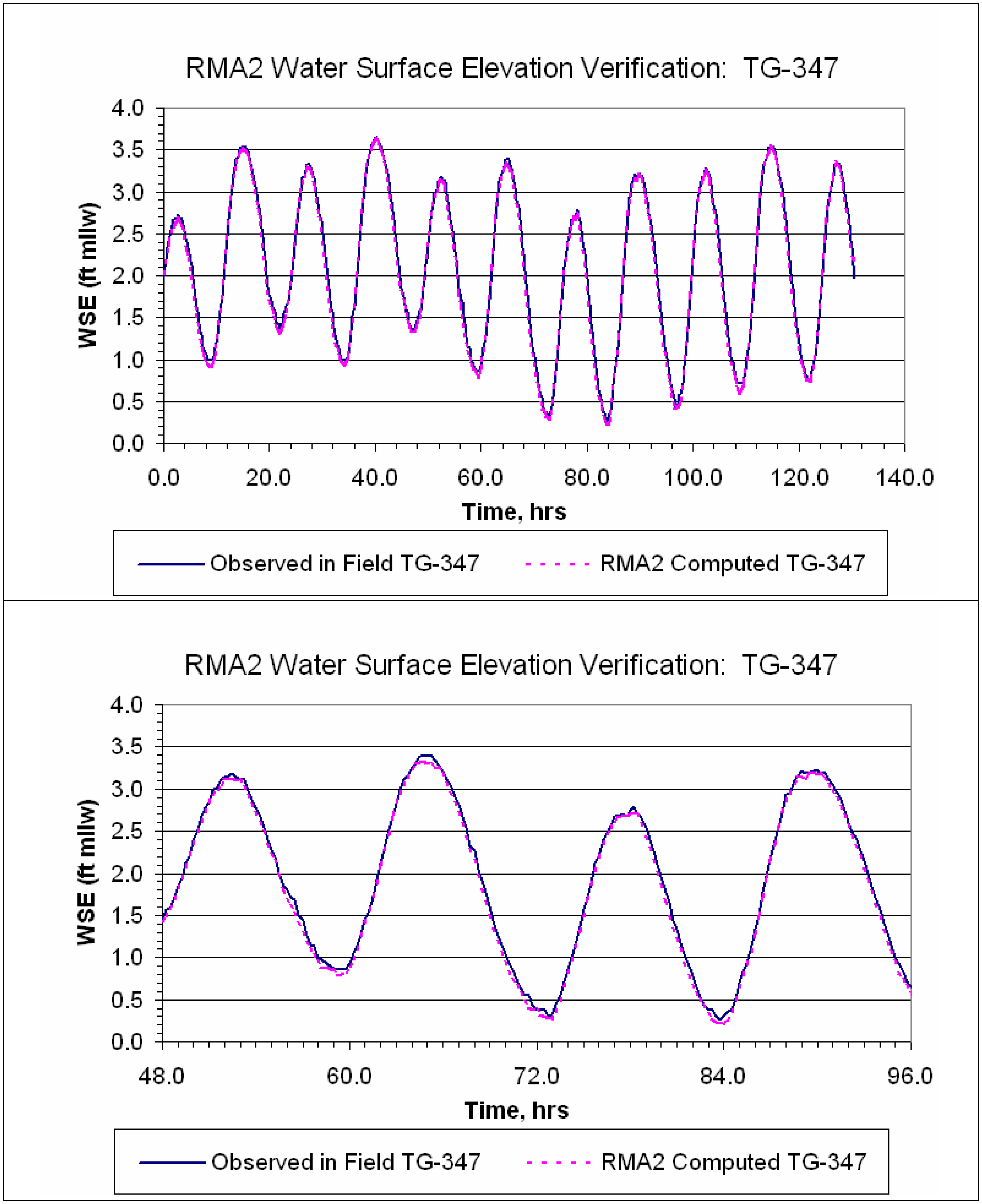


Figure 60. Water surface elevation (WSE) measurement at field tide gage 347 versus RMA2 computations.

The statistical comparisons of model versus measured water surface elevation use the statistical measures described above. The results of the statistical error measures are presented in Table 12. These measures are quite satisfactory.

Table 12. Statistical error measures for the model at the two water surface tide gages.

Gage	MAE, ft	RMSE, ft	COE	COD
TG-346	0.107	0.139	0.990	0.991
TG-347	0.058	0.069	0.997	0.998

Velocity verification

For the August 2001 13-hr survey, there was no stationary point velocity gage. Instead, the survey boat ran survey lines at 1-hr intervals with ADCP equipment and measured the water speed and direction for numerous water depth profiles. Representative snapshots of that data were chosen to represent each maximum flood and ebb event around the 400-ft-wide gap opening, and were reduced in HyPAS to represent a "depth-averaged" condition. This reduction was required in order to accurately compare these to RMA2, which is a depth-averaged solution. Using this method, it is imperative to have the same vector scaling for both data sets. Figures 61 and 62 are velocity vector plots of the maximum flood and maximum ebb events, respectively. The yellow vectors represent the RMA2 solution, while the blue vectors are from the depth-averaged ADCP survey line. Unlike the statistical measures, this type of comparison is "subjective." The agreement appears very good for the maximum flood vectors. However, there is slight disagreement in the vicinity of the northern gap for the maximum ebb eddy pattern.

Conclusions

The RMA2 model results have compared very favorably to the August 2001 field data set. The hydrodynamic model results can now be confidently used to examine general circulation and flushing characteristics of the harbor.

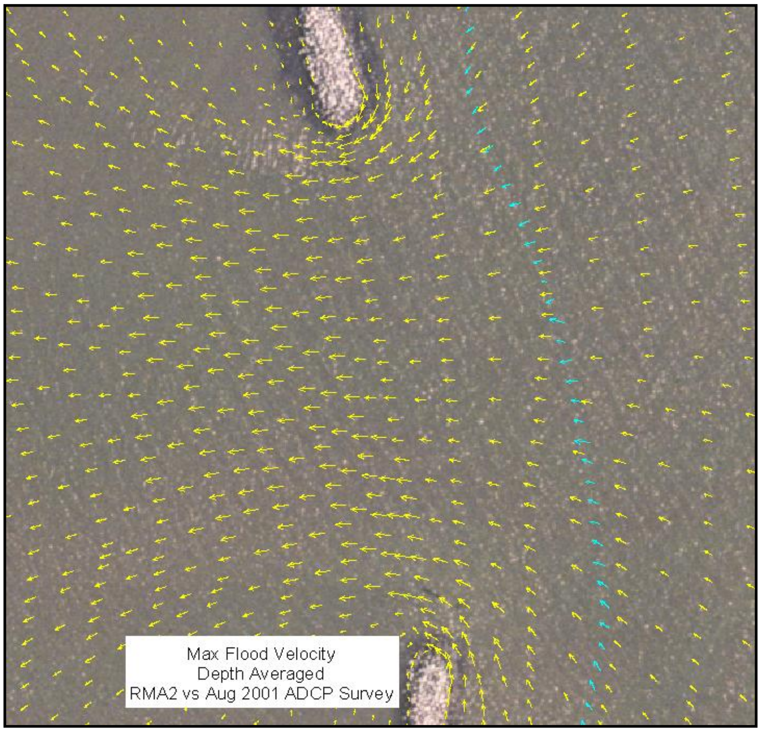


Figure 61. RMA2 velocity vectors (yellow) compared with ADCP depth-averaged field vectors (blue) at maximum flood condition.

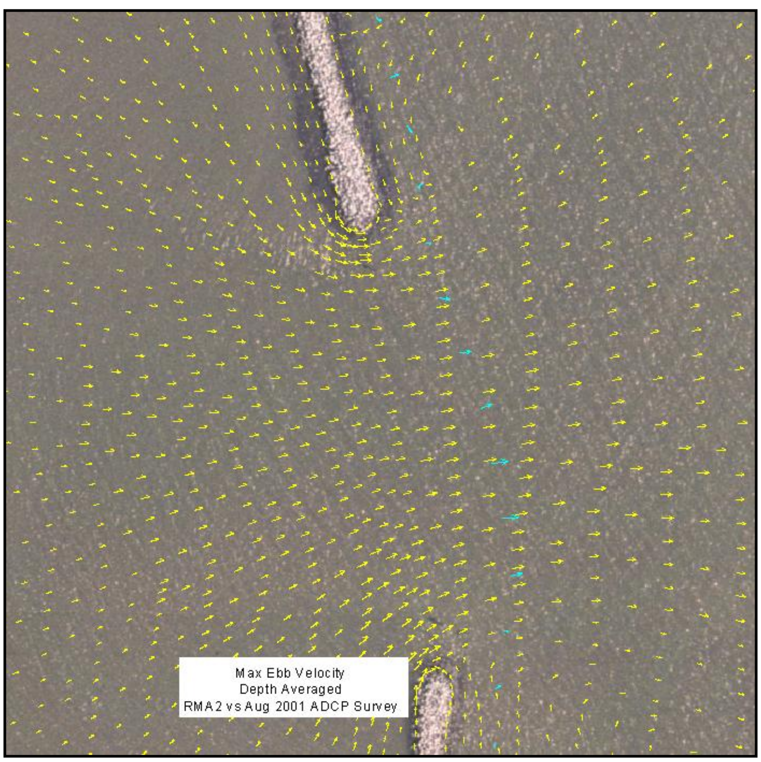


Figure 62. RMA2 velocity vectors (yellow) compared with ADCP depth-averaged field vectors (blue) at maximum ebb condition.

General circulation patterns

Although no aerial flow trace field data exist, SMS has the ability to post-process vector patterns, flow traces, and drogue plots for either a steady-state event or for a transient simulation of the RMA2 solution. These tools can give insight to understanding the general circulation patterns and flushing characteristics of the harbor.

The most direct method of understanding the circulation pattern is with an SMS velocity vector plot, again concentrating on the portion of the simulation where maximum ebb and maximum flood currents occur. Figure 63 shows the RMA2 scaled velocity vectors and color contours of velocity magnitude at the time of maximum ebb. The RMA2 nodal solution vectors are scaled and displayed on a regular grid for ease of interpretation. Figure 64 shows the velocity pattern for maximum flood. Figure 65 plots the time series directional currents and the water depth for hours 50 to 100 of the RMA2 simulation for a point located in the central gap, halfway between the two structures. The exact location of the plotted time series data is shown in Figures 63 and 64 as a gray circle located in the navigation channel of the central gap. The red dashed vertical lines in Figure 65 mark the hours that maximum flood and maximum ebb circulation patterns are presented in Figures 63 and 64.

The SMS flow trace depiction of the RMA2 velocity solutions at time of maximum ebb and maximum flood are shown in Figures 66 and 77, respectively. The ebb flow trace depicts a straightforward emptying of the harbor. However, the flood flow trace has a complex set of intricate eddy patterns in the shallow regions of the harbor. The SMS flow trace animation parameter settings for these figures are (a) number of particles per object to be distributed over the domain = 1, (b) decay ratio of the particle tail = 0.5, (c) average particle speed to magnify the activity with the visual scene = 0.25, (d) flow trace limit or the maximum distance a particle can travel in a single integration step = 12, and (e) velocity difference limit which governs how quickly a particle can change direction = 1. The parameters are adjusted until the best visual is obtained, but the actual magnitude of velocity is not available with a flow trace plot.

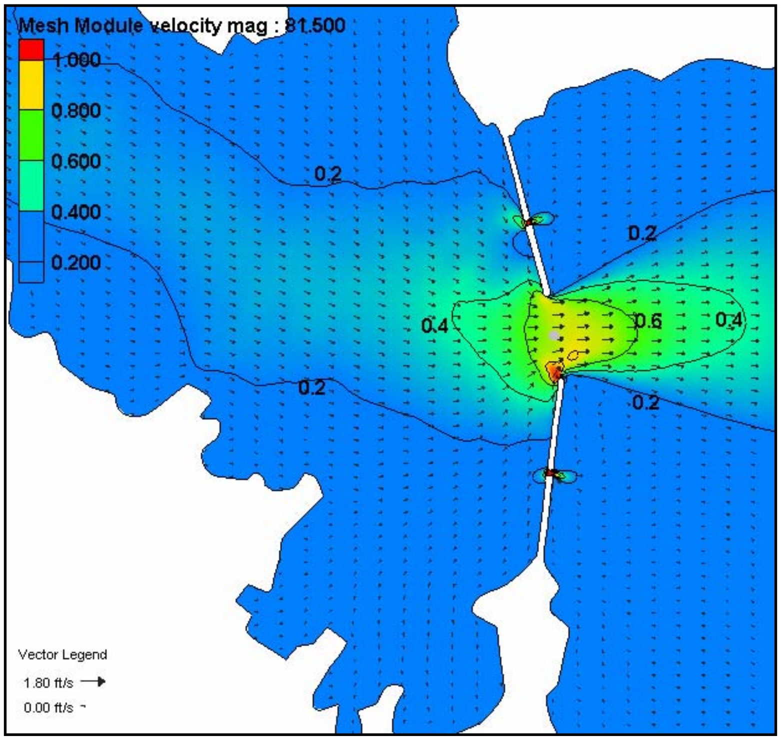


Figure 63. RMA2 snapshot of the study area for time of maximum ebb velocity within the central gap (simulation time = 80.25 hr).

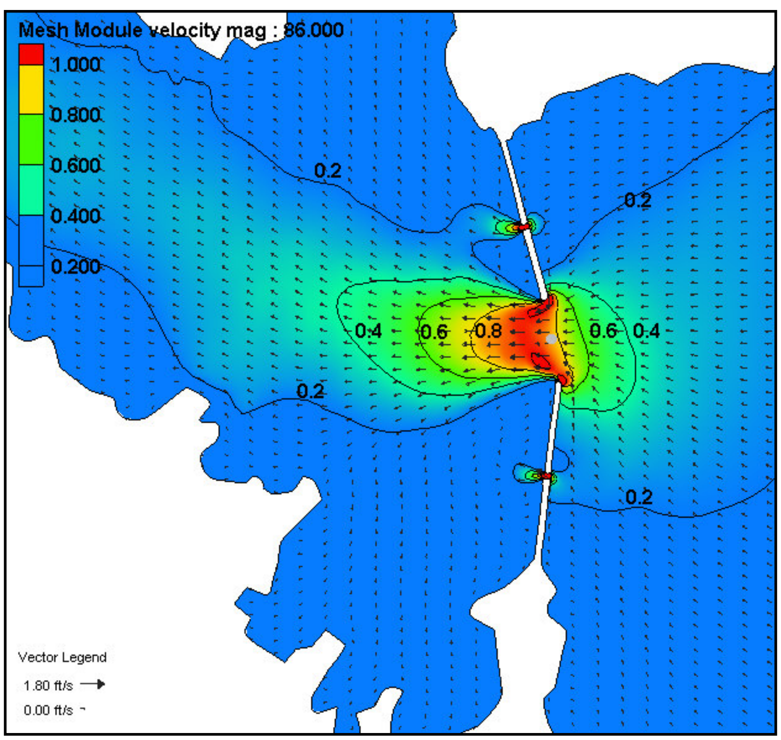


Figure 64. RMA2 snapshot of the study area for time of maximum flood velocity within the central gap (simulation time = 86.0 hr).

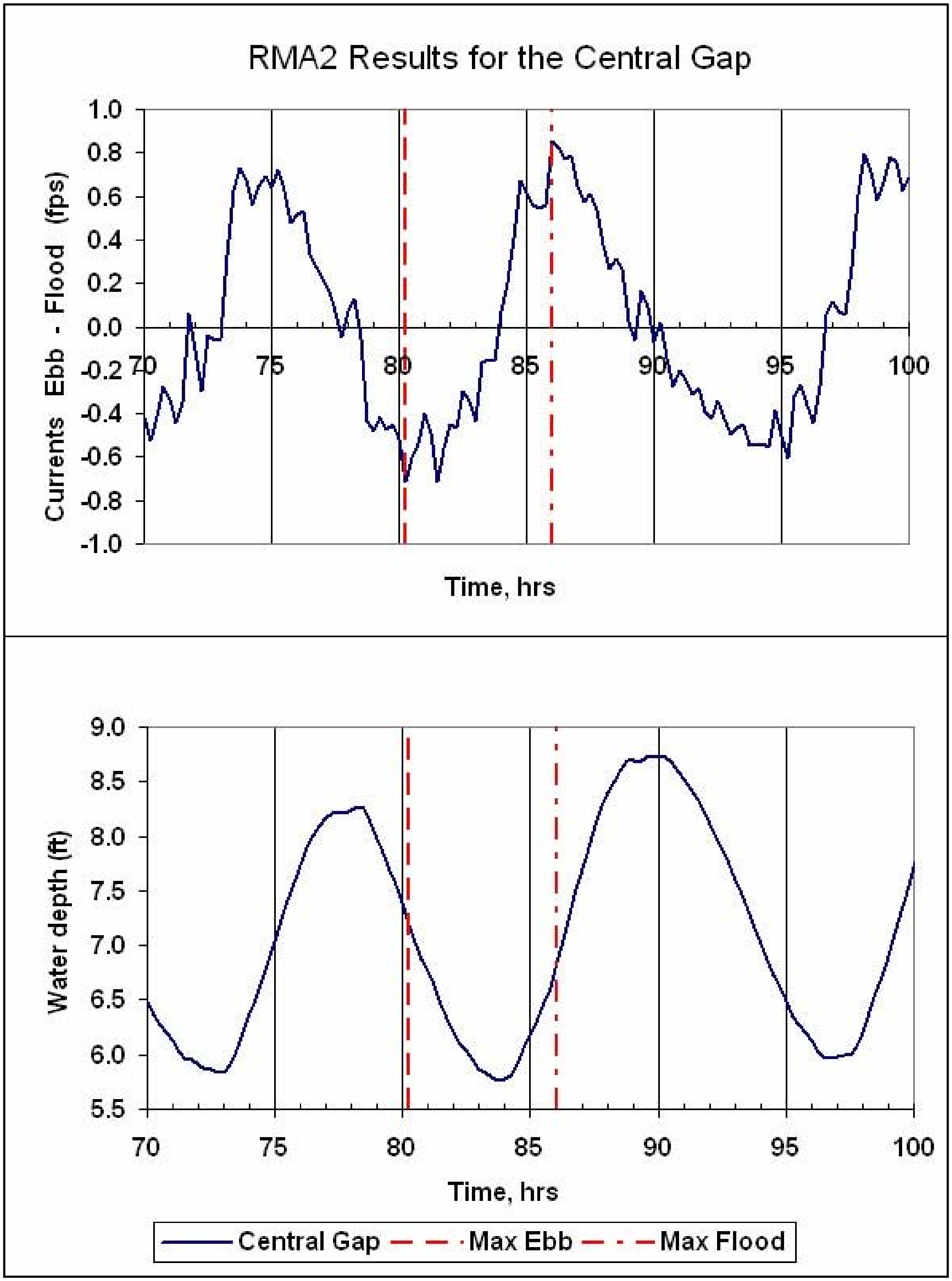


Figure 65. Examination of RMA2 results at mid-point of the central gap.

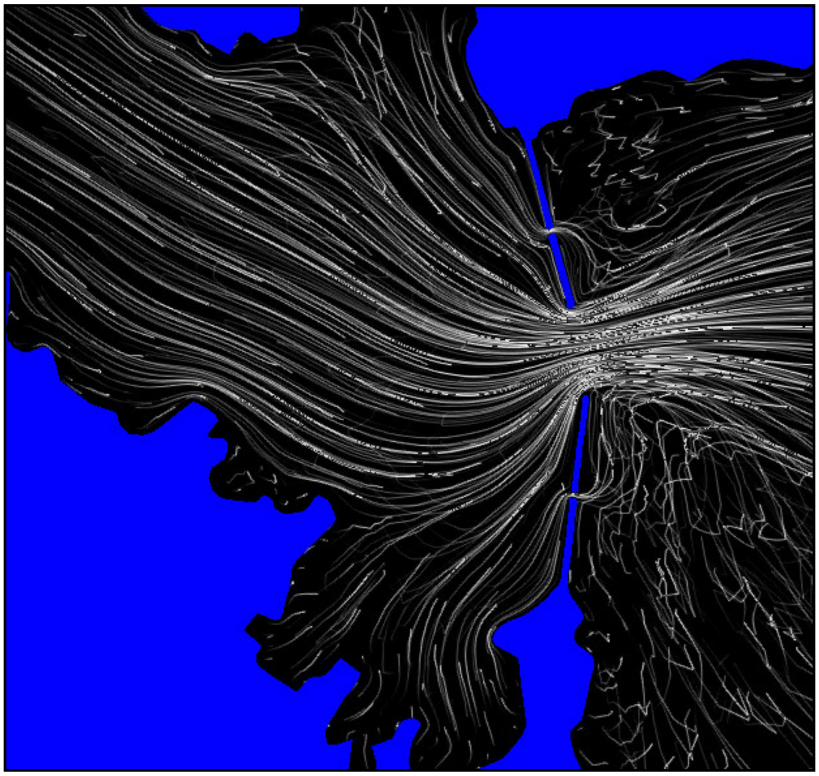


Figure 66. SMS flow trace of RMA2 velocity solution at time of maximum ebb.

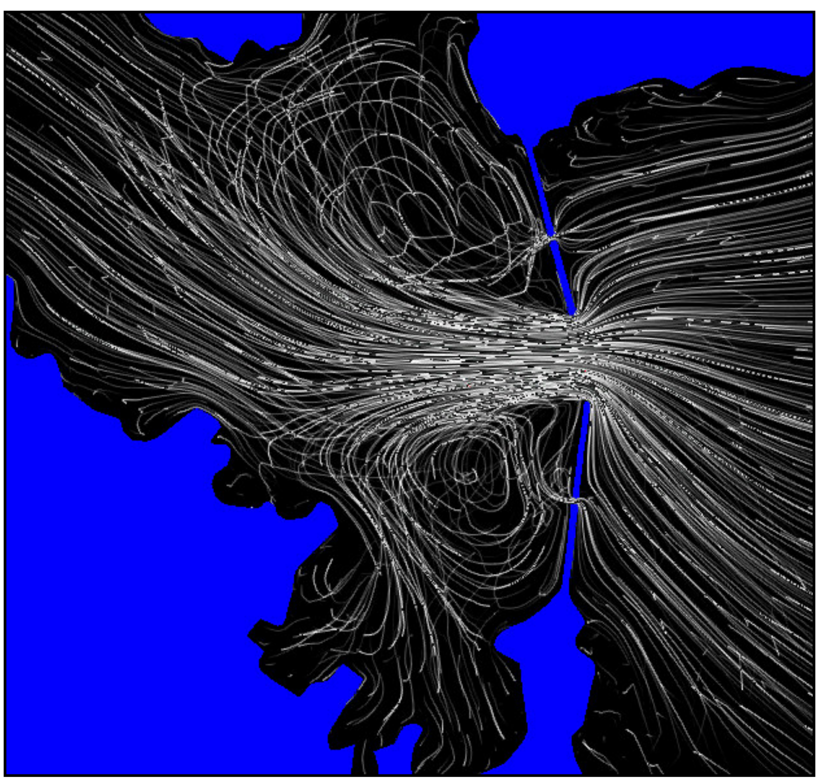


Figure 67. SMS flow trace of RMA2 velocity solution at time of maximum flood.

The drogue plot capability within SMS allows a set of "drogue points" to be placed within the computational domain and allowed to "float" with the currents. The SMS drogue animation parameters for these samples are (a) size of the head = 5, (b) length of time before the tail fades = 3 hr), and (c) ability to change color based on velocity is >15 cm/sec (>0.50 ft/sec).

A representative set of drogues was placed within the numerical solution at time = 30.5 hr as shown in Figure 68, and carried with the currents for as much as 100 hr (time = 130.5 hr). A frame was created every 2 hr of the simulation. The drogues on the tidal side of the structures are approximately 17 m (55 ft) apart. The drogues spanning the width of the upper harbor were spaced approximately 30 m (100 ft) apart. The locations of the public pier and the boat dock are labeled for reference. For drogue animations, if a particle floats out of the visual domain shown in Figure 68, then that particle (drogue) cannot re-enter the scene. The drogue locations highlighted in white never left the visual domain.

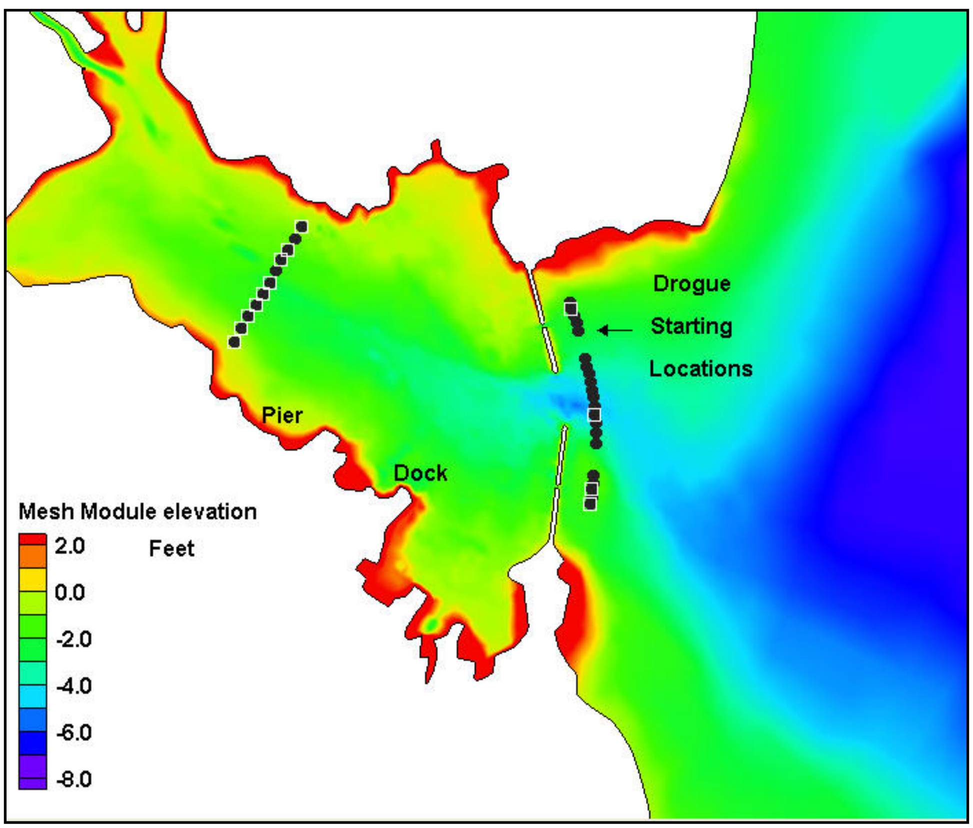


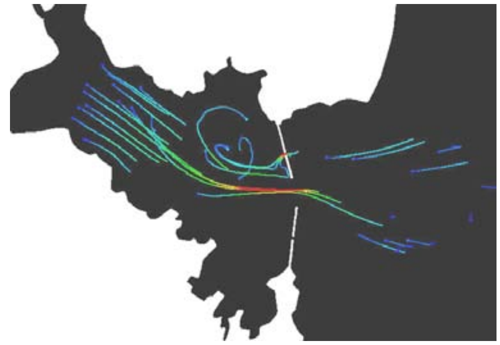
Figure 68. Starting locations for the drogue animation, with the bottom elevation bathymetry shown as the background.

To illustrate the results of the drogue animation, a still shot was captured at a three-frame frequency (every 6 simulation hours). Several frames are presented in sequential order in Figure 69, then skipping to conclude with the last frame. The water surface elevation signal is provided as a quick reference. A red color on a drogue tail means the velocity exceeded 15 cm/sec (0.5 ft/sec) for that travel period.

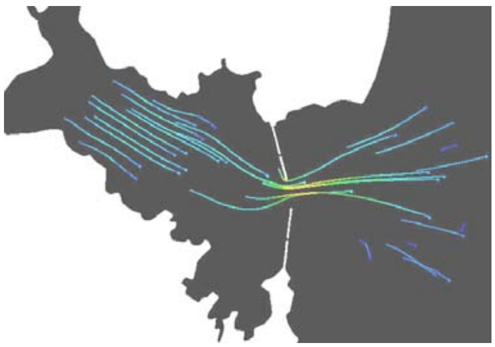
As seen in Figure 69 at the conclusion of the animation, frame 50 only had 13 out of the original 32 drogues remaining active in the scene after 100 hr of tidal response. This illustrates that the harbor is flushing to some uncertain degree.

Another method to examine general circulation patterns in a tidal environment is to analyze the “net velocity” over one or more complete tidal cycles. For the August 2001 verification period, it is difficult to isolate a true tidal cycle that is dominated by the M2 harmonic tidal component (period = 12.42 hr). As seen in Figure 58, the red box outlines hours 13.0 to 124.75, which represent 111.75 simulation hours or 8.99 M2 tidal cycles. An ERDC CHL utility called “mergeave” that comes with the TABS-MD suite of numerical models was used to create an integrated or averaged solution for the computational domain. This solution was then read into SMS where net velocity vectors were plotted at several magnifications, as shown in Figure 70. The vector scales were uniquely magnified for each zoom to emphasize patterns.

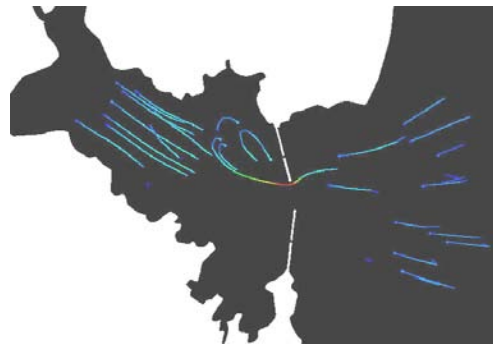
Frame 3, hr = 34.5, ebb



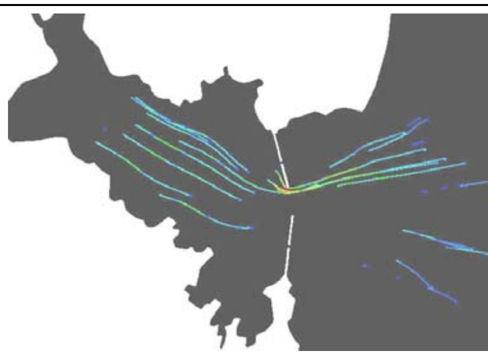
Frame 6, hr = 40.5, flood



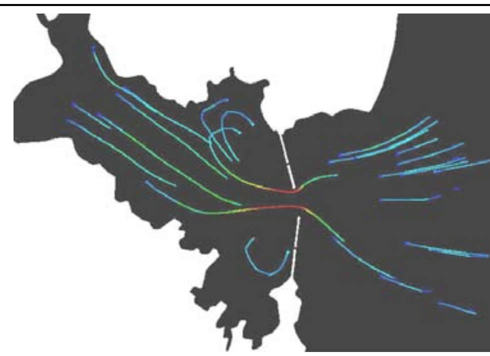
Frame 9, hr = 46.5, ebb



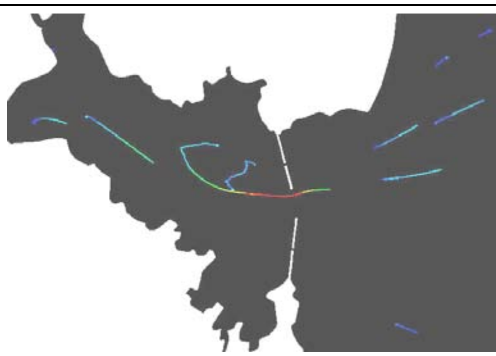
Frame 12, hr = 52.5, flood



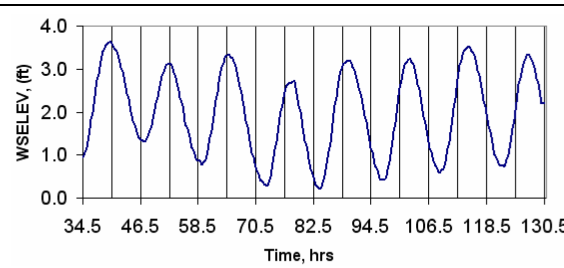
Frame 15, hr = 58.5, ebb



Frame 18, hr = 64.5 flood



Frame 50, hr = 130.5, end of simulation



Central gap water surface elevation reference.

Figure 69. Selected frames from the drogue animation.

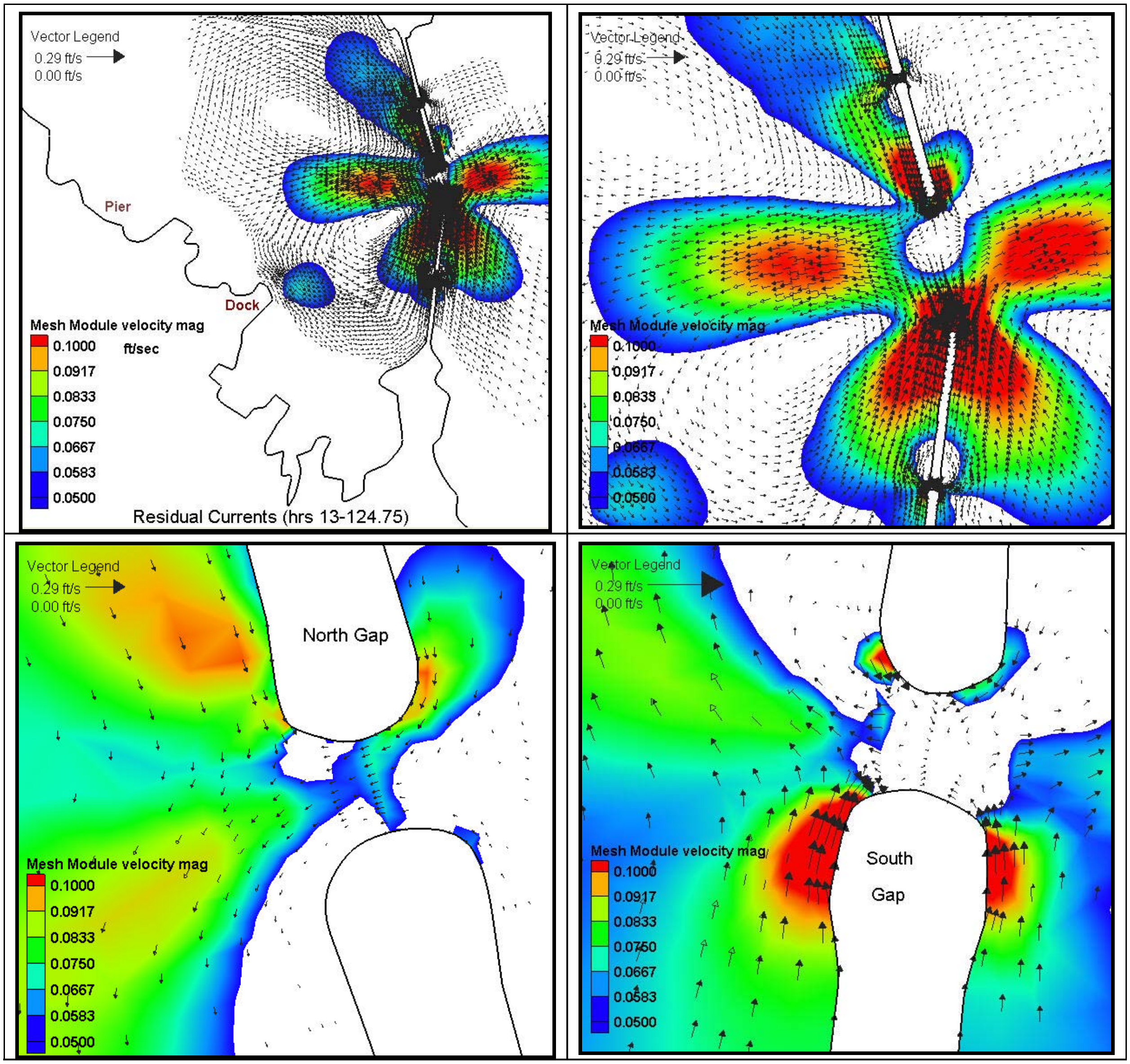


Figure 70. Various views of residual velocity vector patterns in the vicinity of the three gaps.

The rather unusual residual currents displayed are caused from the asymmetry in the velocity patterns during strength of flood and strength of ebb flow. When the water needs to pass through the narrow gap in the jetty there will be a “gathering” flow pattern similar to flow through a funnel as the water approaches the gap. As the flow passes through the gap, there will be a jet type of flow that generates the eddies on the sides. When these patterns are averaged, the stronger jet side of the flow will dominate the patterns. Only the small northern gap shows a slight flood residual dominance.

So far, none of the methodologies quantify the extent to which the as-built existing condition harbor has the ability to flush. To address this issue, an RMA4 conservative constituent transport numerical simulation was performed.

RMA4 – Two-dimensional water quality constituent transport model

RMA4 is a finite element water quality constituent transport numerical model in which the depth concentration distribution is assumed uniform. RMA4 resides within the TABS-MD system and is designed to use the RMA2 hydrodynamic solution as input. RMA4 was originally developed by Norton et al. (1977) of Resource Management Associates, Inc., Davis, CA. Modifications to the original code have been made by a number of CHL researchers.

RMA4 is designed to simulate the depth-averaged advection-diffusion process in an aquatic environment. The model can be used for the evaluation of any conservative substance that is either dissolved in the water or that may be assumed to be neutrally buoyant within the water column. A complete user's guide documentation of RMA4 is available for download from
http://chl.erdc.usace.army.mil/chl.aspx?p=s&a=Software!15

The serial (frontal) version of RMA4 version 4.54 was used for this Windows personal computer application.

The goal of this RMA4 simulation was to conduct experiments to determine if the Tedious Creek Harbor flushed a contaminant within a reasonable period of time. The first of these experiments initializes the entire numerical domain to a zero concentration, and then drops a conservative contaminant near the northern edge of the boat dock. The contaminant would issue its load for 4 hr, and then be turned off. The incoming tidal waters would be tagged as fresh water. The flood/ebb hydrodynamic cycle would be repeated until the flushing test concluded.

RMA4 run control

The RMA4 model is interfaced with the SMS for graphics and for efficient implementation of pre- and post-processing. Setting up RMA4 is rather straightforward because it uses the same mesh geometry file (*.geo) and

the RMA2 hydrodynamic solution, previously described, as input. Many of the verified RMA2 parameters will carry over to the RMA4 simulation.

Scaling control

Since RMA4 uses metric units, the English geometry and English RMA2 hydrodynamic solution inputs were scaled to metric with the GS- and HS-Cards as shown below.

```
GS  0.3048  0.3048
HS  0.3048  0.3048  0.3048
```

Timing control

The timing control for the RMA4 model is assigned on the TC-Card and TH-Cards. The TC-Card sets the model time step, total number of time steps, and maximum time of the simulation. The TH-Card defines the hours to skip at the beginning, and the last hour to use from the RMA2 solution. Again, refer to Figure 58 where the red box outlines hours 13.0 to 124.75, which represent 111.75 simulation hours or 8.99 M2 tidal cycles. This segment of the RMA2 simulation will be used for two reasons: it eliminates the hydrodynamic model spin-up period, and it represents a set of M2 tidal cycles that may be used repetitively if necessary. Consequently, RMA2 hour 13.0 will be RMA4 hour 0.0.

```
CO .. TSTART  DELT   NSTEP  TIMEMAX  SSF
TC     0.0      0.25   448     111.75    1
CO .. Time Control for RMA2 hydro (hrs to skip-ending hr)
TH    12.99    124.75
```

Other parameters

This simulation will only transport one constituent (FQ-Card) and will be treated as conservative with zero decay (FQC-Card). The dispersion coefficients for this RMA4 simulation will be controlled with the Peclet formulation, consistent with the strategy used in the RMA2 verification.

```
CO.. NQUALITY CONSTITUENTS (NQAL=1)
FQ   1 0
CO.. QUALITY DECAY CONTROL (DECAY COEFF=0)
FQC  0.0
CO ... RMA4 has Same Peclet control as in RMA2 (with unit conversion to meters)
PE  1  10  0.3048   1  1
```

Note that in RMA4 the Peclet number is used to set a mixing coefficient rather than the eddy viscosity, based on

$$P_e = \frac{VL}{D_x} \tag{6}$$

where D_x is the turbulent mixing coefficient in units of m^2/sec.

RMA4 Flushing Demonstration No. 1

For the first flushing demonstration, the Tedious Creek computational domain (2,822,064.2 m^2 (30,376,446 $ft^{2)}$)) was initialized to 0.0 parts per thousand (ppt) everywhere. A small element (element no. 2399, area = 29 m^2) situated near the tip of the boat dock was selected as the load location. As illustrated in the example, the boundary condition input mass load is defined with a BLE-Card. An input value of 5 g/sec (0.18 oz./sec) uniformly over 1 element for (4 hr x 3,600 sec/hr) yields a total load of 72,000 g (159 lb) of contaminant. The inflow tidal boundary, defined with a BCL-Card, was assigned zero concentration for inflow. The END-Card functions to mark the conclusion of data for a time step and as a comment holder.

```
BLE  2399   5.0
BCL  1   0   1   .5
END - -   Time=    0.00  Step=   0  <<load turned on for 4 hr
END - -   Time=    0.25  Step=   1
END - -   Time=    0.50  Step=   2
END - -   Time=    0.75  Step=   3
END - -   Time=    1.00  Step=   4
END - -   Time=    1.25  Step=   5
. . . skip
END - -   Time=    3.75  Step=  15
END - -   Time=    4.00  Step=  16
BLE  2399   0.0
BCL  1 0    1 .5
END - -   Time=    4.25  Step=  17  << Turn off load @ hr 4.25
END - -   Time=    4.50  Step=  18
. . .  skip to end . . .
END - -   Time=   150.00  Step= 600
STOP
```

The RMA4 spill demonstration simulation ran to completion in less than 9 min of clock time on a Dell Dimension 8250 with a 2.66-GHz Pentium chip and 512 MB of RAM.

The ideal way to view RMA4 results is to use SMS to generate an animation of the contaminant plume. The first frame of the animation is shown in Figure 71. The color scale is red for a high concentration of contaminant and blue for 0.01 ppt. A “white” coloring means that part of the domain is less than 0.01 ppt. The outline of the computational domain is shown with the contaminant load resulting from the first time step near the dock. To illustrate the results of the contaminant plume animation, a still shot was captured at an increasing frame frequency (every 10 frames is approximately every 5.0 simulation hr). Selected frames are presented in sequential order in Figures 72 – 74, then skipping to conclude with the last frame.

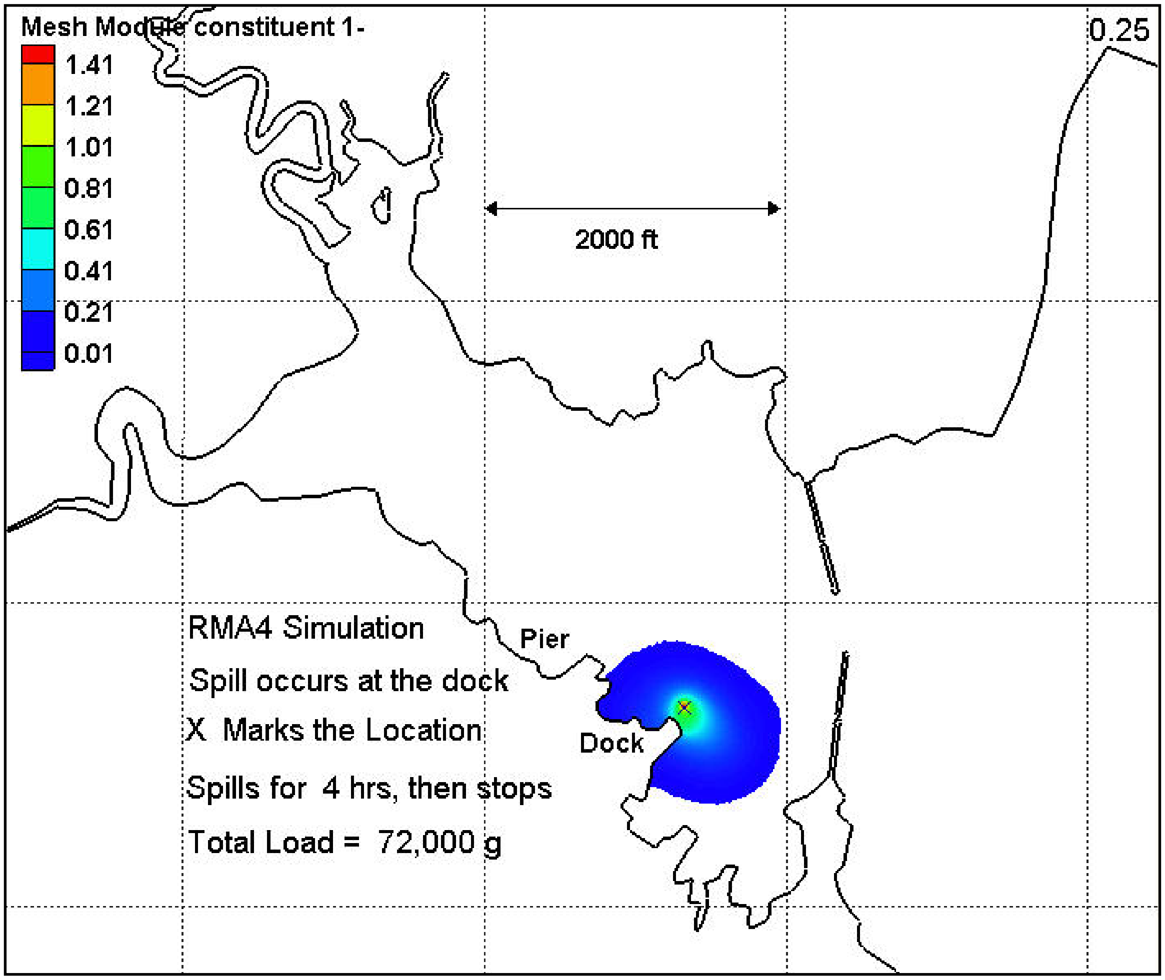
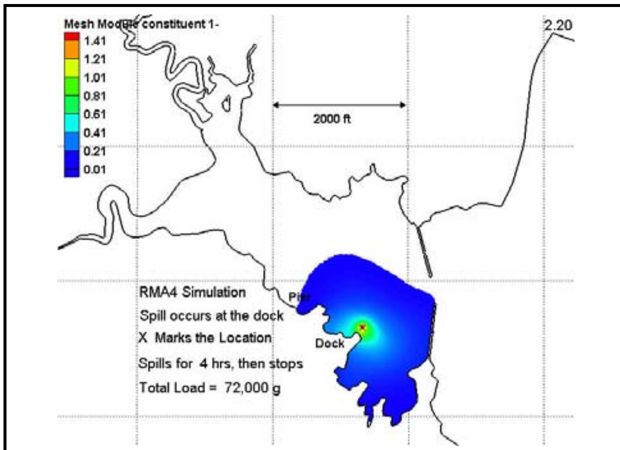
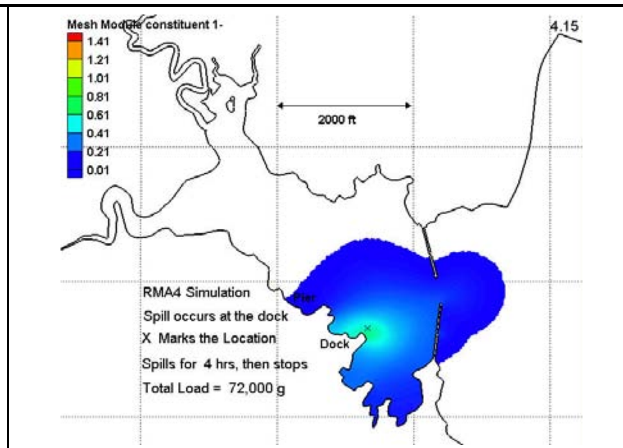


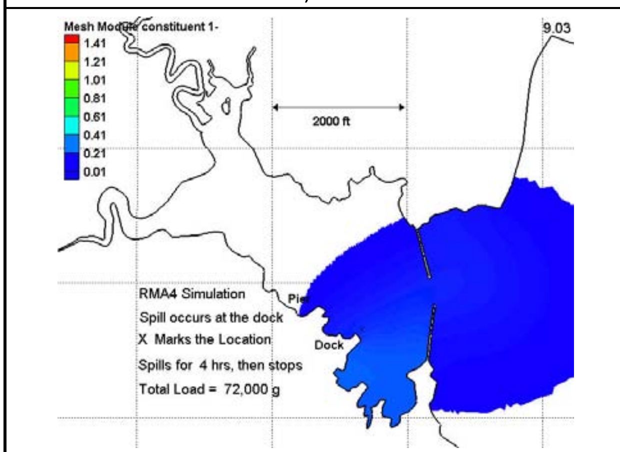
Figure 71. First time step of the RMA4 constituent transport spill simulation (time = 0.25 hr).



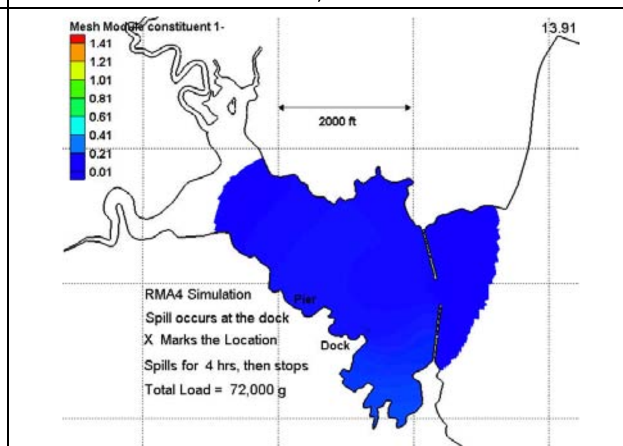
Frame 6, hr = 2.20



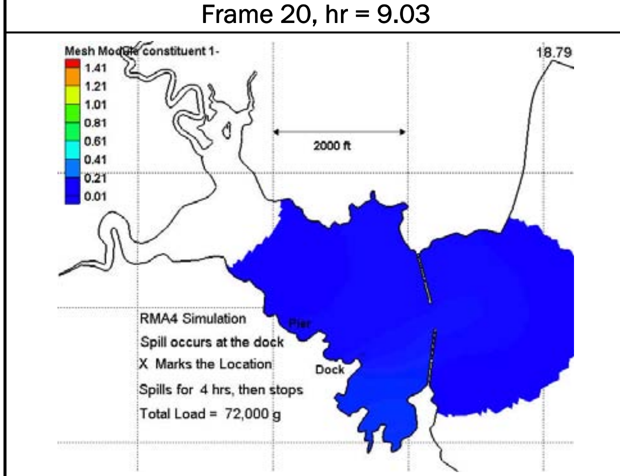
Frame 10, hr = 4.15



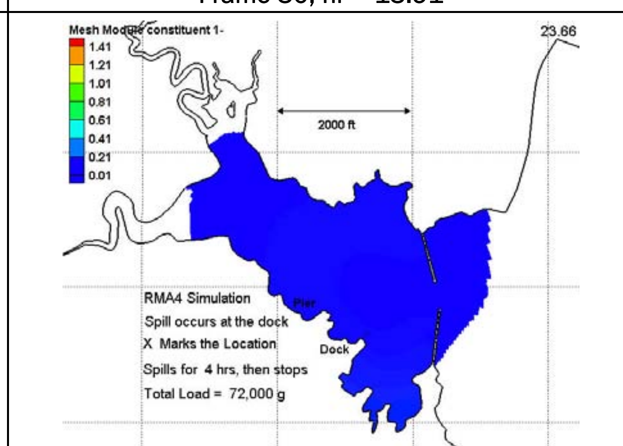
Frame 20, hr = 9.03



Frame 30, hr = 13.91



Frame 40, hr = 18.79



Frame 50, hr = 23.86

Figure 72. Still frames of the RMA4 constituent transport spill simulation (frames 4-50).

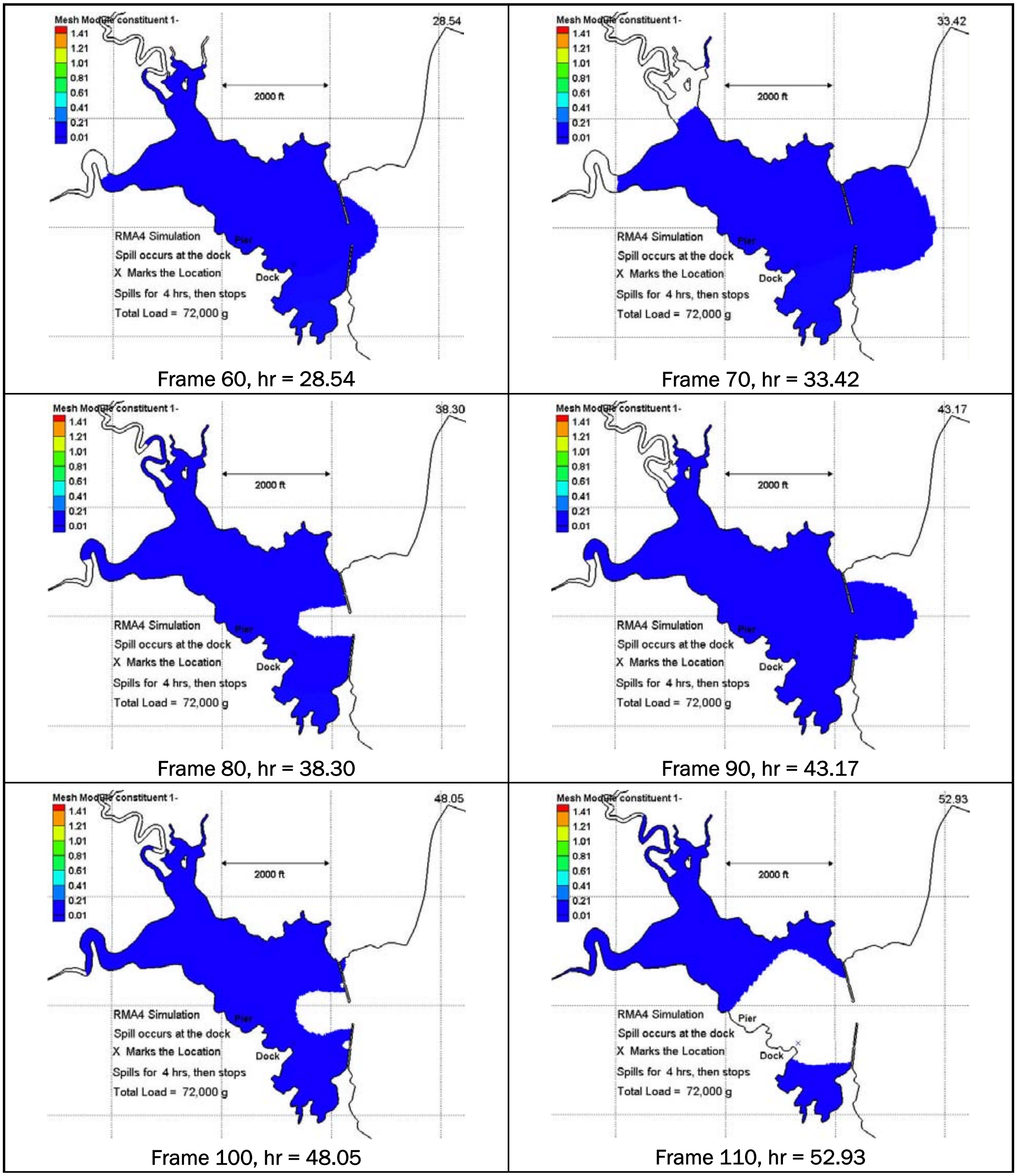


Figure 73. Still frames of the RMA4 constituent transport spill simulation (frames 60-110).

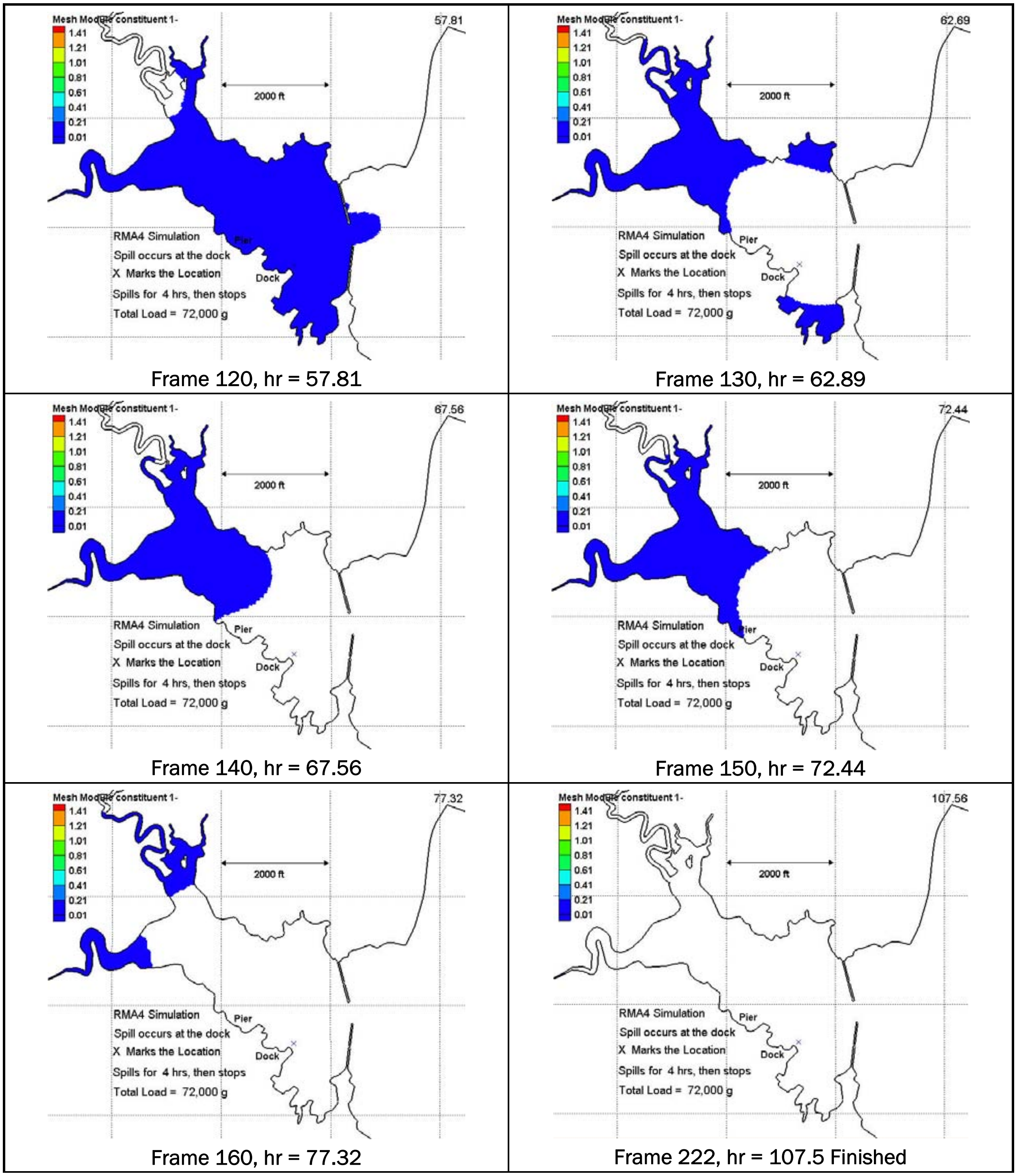


Figure 74. Still frames of the RMA4 constituent transport spill simulation (frames 120-end).

This simulation demonstrates that the contaminant spilled near the Tedious Creek boat dock flushes within 107.5 hr.

RMA4 Flushing Demonstration No. 2

For the second flushing demonstration, the Tedious Creek domain was initialized to 1.0 ppt everywhere except east of the structures, which was initialized as fresh (0.0 ppt). The initial conditions were applied with IC-blank, ICT, and ICE-Cards. The inflow tidal boundary, defined with a BCL-Card, was assigned zero concentration for inflow. The END-Card functions to mark the conclusion of data for a time step and as a comment holder.

```
BCL  1   0   1   .5
END - -   Time=     0.00  Step=   0
END - -   Time=     0.25  Step=   1
END - -   Time=     0.50  Step=   2
END - -   Time=     0.75  Step=   3
END - -   Time=     1.00  Step=   4
. . .  skip to end . . .
END - -   Time=  1440.00  Step=5760  days= 60.0
STOP
```

The RMA4 flushing demonstration ran 5,760 transient 15-min time steps for a total of 1,440 hr (60 days) of simulation in less than 1.8 hr clock time using a Dell Dimension 8250 with a 2.66-GHz Pentium chip and 512 MB of RAM.

The ideal way to view RMA4 results is to use SMS to generate an animation of the contaminant plume. However, for report purposes, representative still frames are presented in Figures 75 – 77. The first frame of this RMA4 animation corresponds to the first time step, time = 0.25 hr, as shown in Figure 75. The color contours range from fresh (blue) to contaminated (red). A “white” coloring means that part of the domain is below 0.01 ppt. Frames were initially collected more frequently to illustrate the dynamics of the tidal harbor, then every 100 hr up to the conclusion of the 60-day simulation.

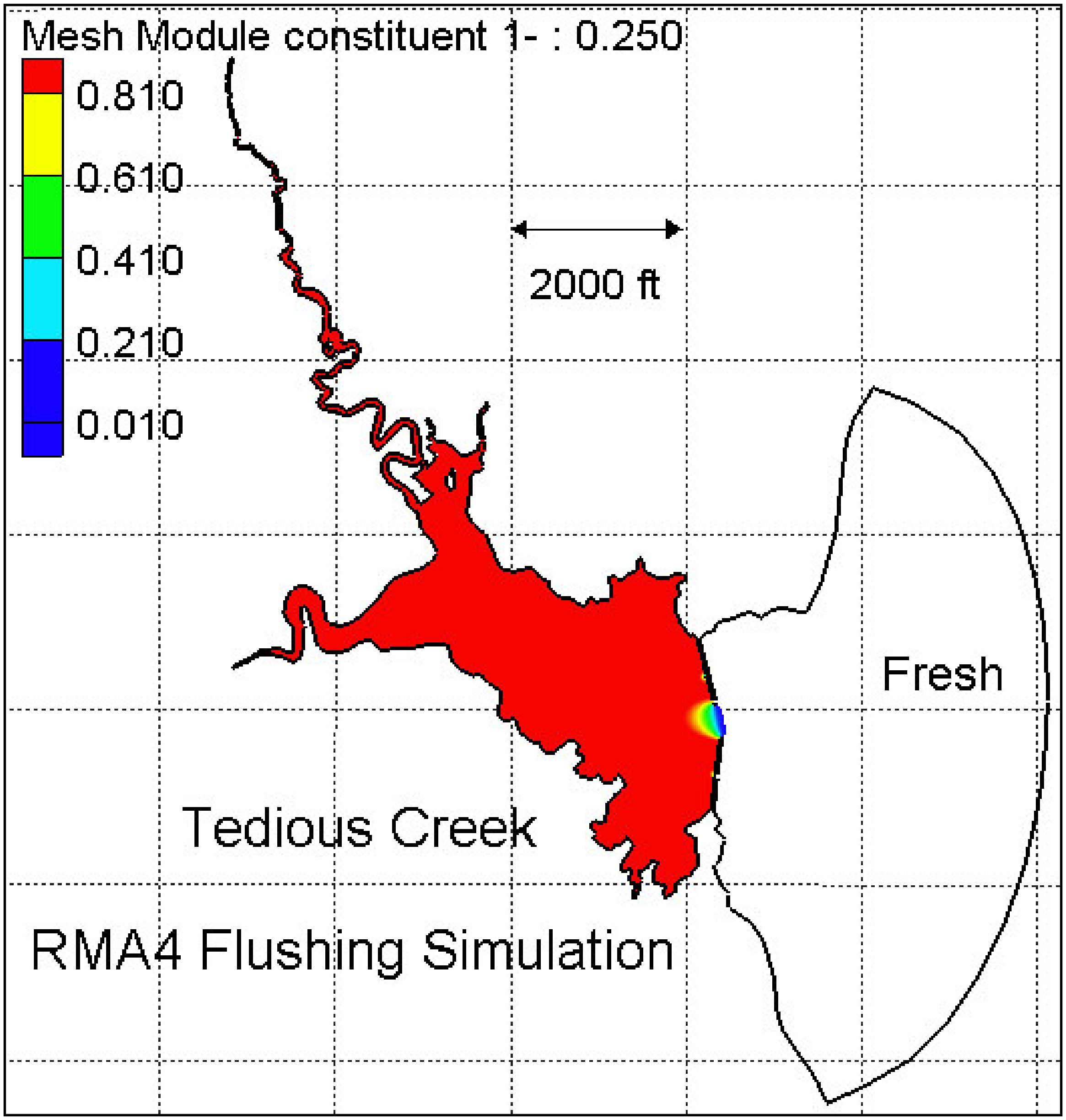


Figure 75. First time step of the RMA4 harbor flushing simulation (time = 0.25 hr).

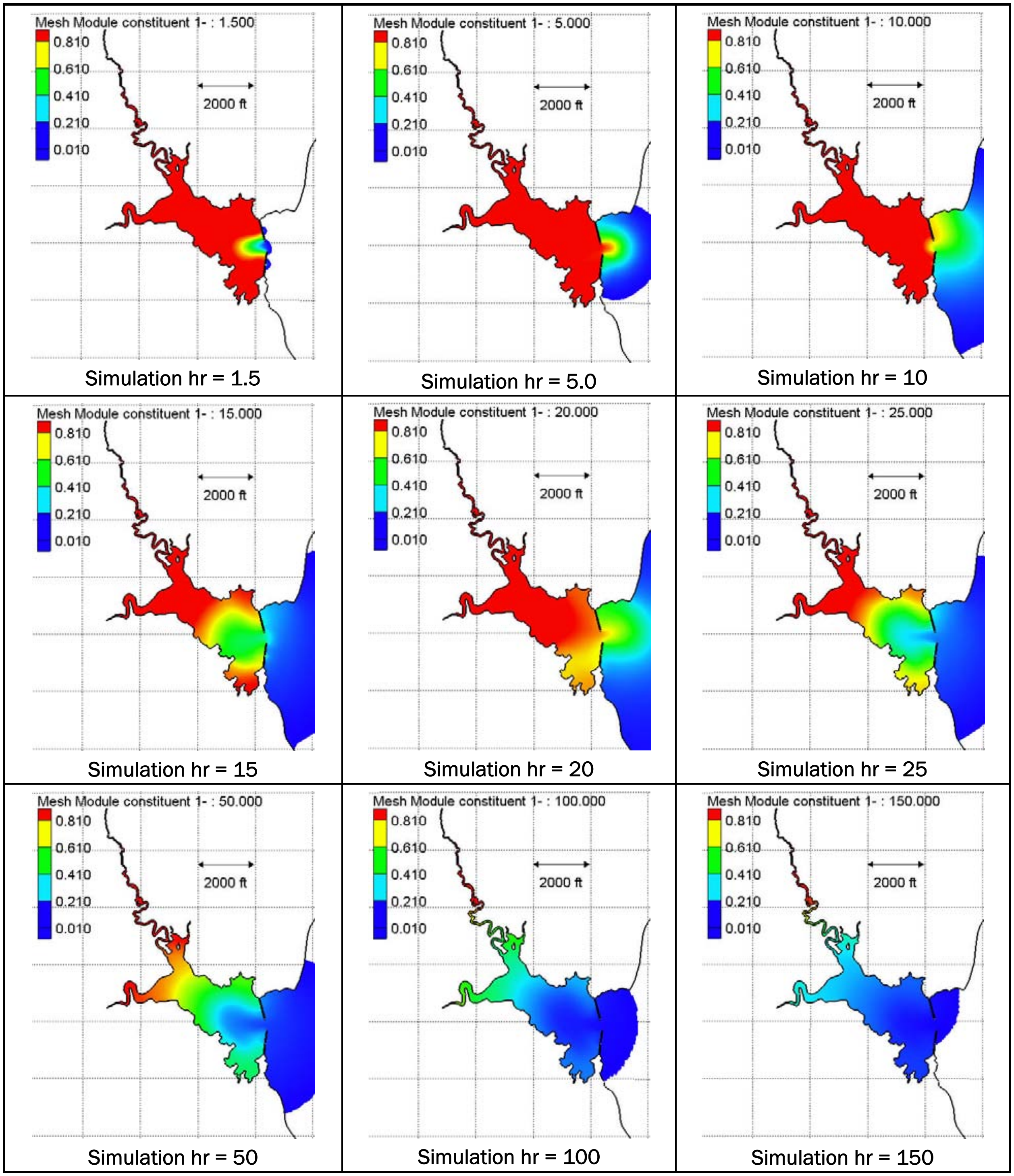


Figure 76. Still frames of the RMA4 constituent transport harbor flushing simulation (time = 1.5 to 150 hr).

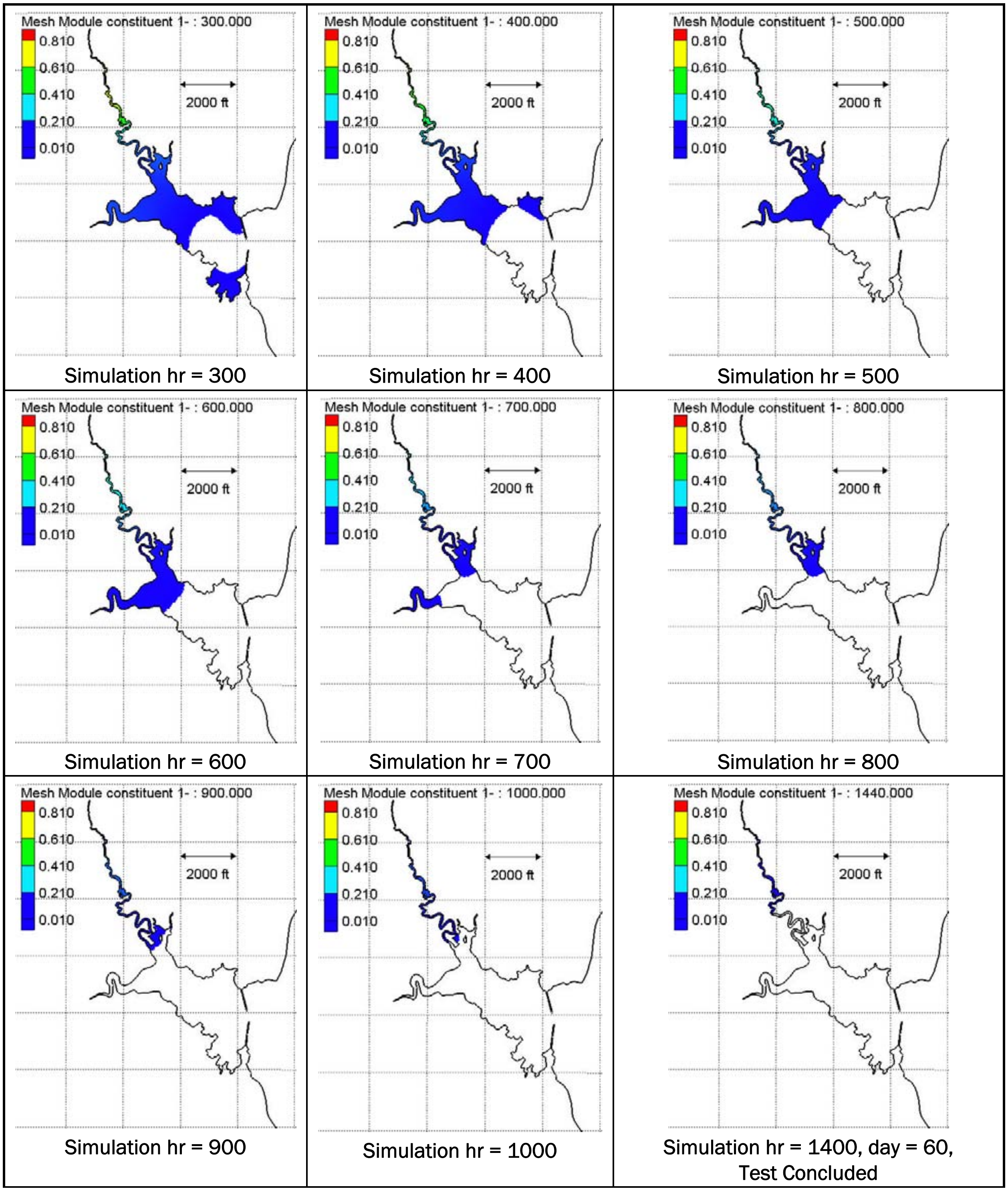
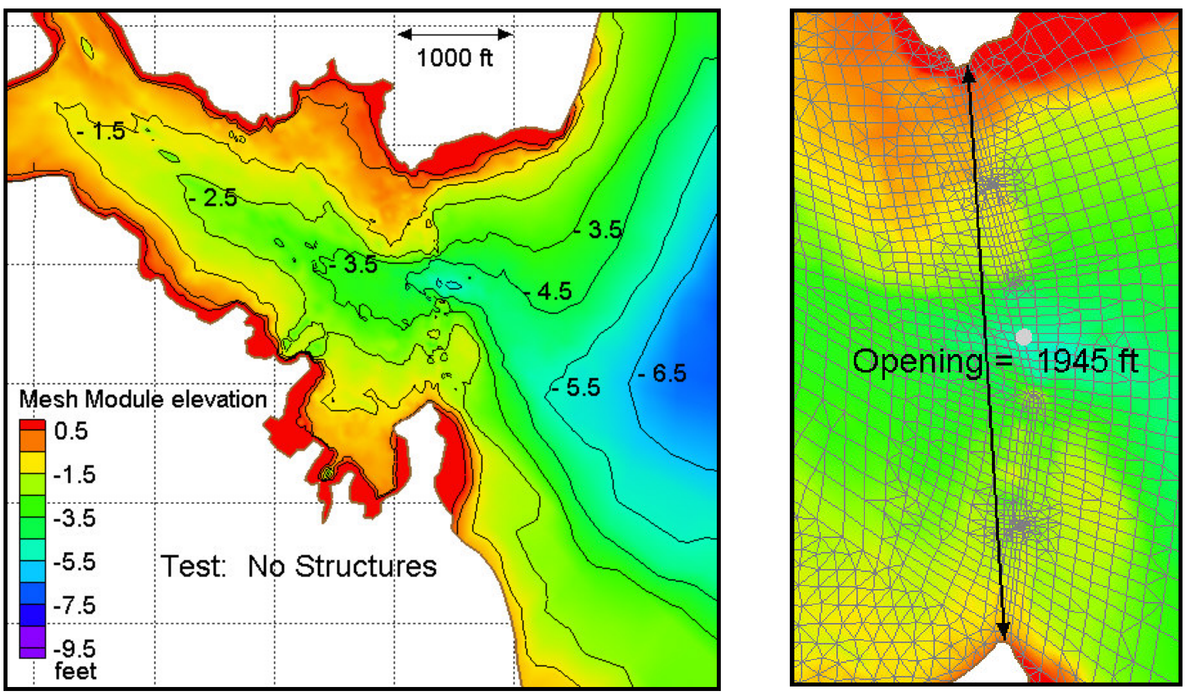


Figure 77. Selected frames of the RMA4 constituent transport flushing simulation (time = 300 to 1,400 hr (end of test, day 60)).

At the conclusion of the RMA4 harbor flushing test, there was still some contaminant in the 0.2-ppt range in the upper tidal creek. In fact, little cleaning of this tidal creek is evident beyond time = 900 hr.

Effect of the structures

To determine the effects of the structures on the general circulation and harbor flushing, the same verified run control parameters described above were used to test the absence of the structures. The finite element mesh was modified to eliminate the structures and to smooth the bathymetry to harbor depths, as shown in Figure 78. Engineering judgment was applied to lower the bathymetry around the structures to harbor depths.



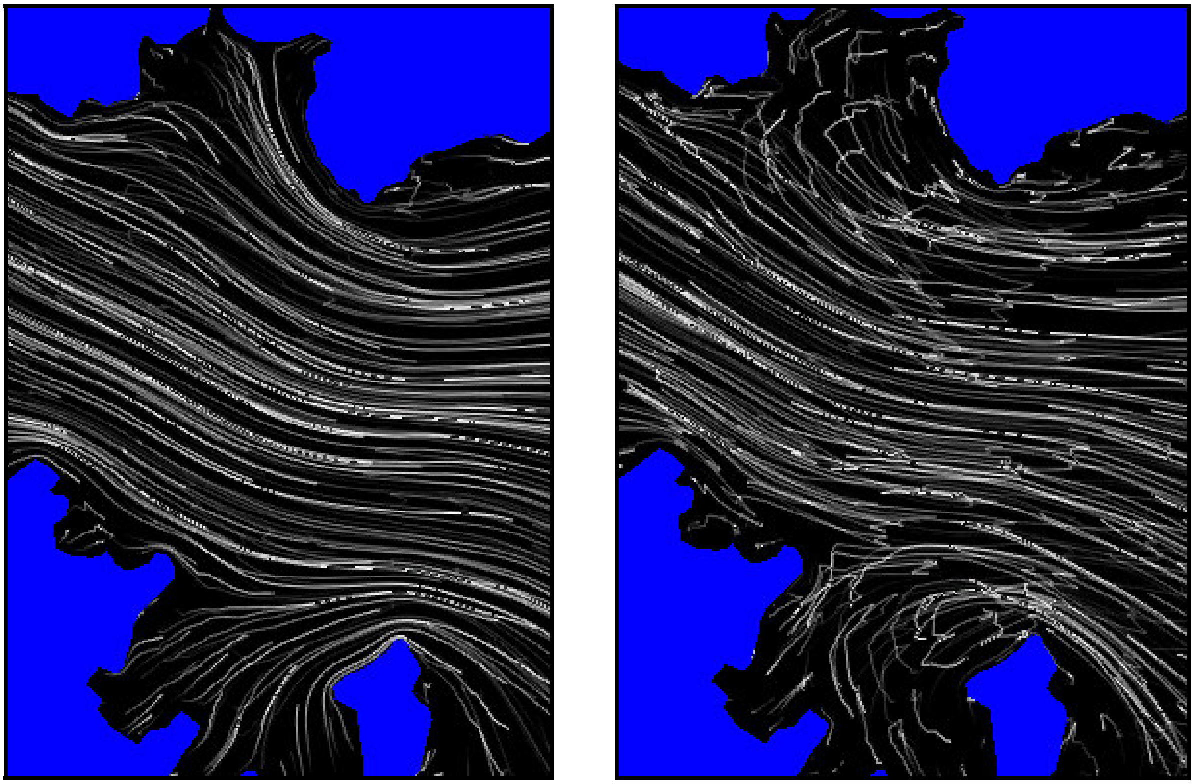
a. Primary study area, structures eliminated

b. Opening details

Figure 78. Tedious Creek bathymetric contours with the structures eliminated; (a) central harbor study area, and (b) opening closeup; August 2001 survey with smoothing to eliminate the structures (elevation feet, NGVD).

Using the no-structure geometry, RMA2 was rerun with verified run control parameters and the August 2001 boundary conditions. As expected, the maximum velocities for the maximum ebb and flood flows in the center of the navigation channel (gray circle in Figure 78b) were lowered from a range of -0.7 to 0.8 fps with the as-built structures to -0.3 to 0.20 fps with no structures. The general circulation patterns for the no-

structure test contained fewer eddy patterns, as shown in Figure 79 (compare Figures 79a and 79b with Figures 66 and 67).



a. Time of maximum ebb

b. Time of maximum flood

Figure 79. SMS flow trace of RMA2 velocity solution (no structures).

Finally, RMA4 Flushing Demonstration No. 1 was rerun using the no structures hydrodynamic solution from RMA2. The results of the flushing test without structures present showed that the 4-hr spill at the dock was cleaned to 0.01 ppt within 131.0 hr. This compares to 107.5 hr for the same flushing demonstration with as-built structures.

Conclusions

The as-built structures appear to maintain good harbor circulation, with velocities below any threat to boats that frequent the harbor. Additionally, the RMA4 flushing tests indicate that the Tedious Creek Harbor has adequate flushing and compares favorably to the no-structure flushing test.

6 Summary and Conclusions

Summary

Tedious Creek is a small, funnel-shaped estuary located on the eastern shoreline of the Chesapeake Bay in Dorchester County, MD. This estuary is located in an area that provides excellent access to many productive fishing grounds in Chesapeake Bay. Tedious Creek Harbor provides anchorage to over 100 vessels involved in commercial and/or recreational fishing.

Problem

Prior to the construction of the breakwater in 1977, the orientation of Tedious Creek allowed the transmission of storm waves that, at times, caused substantial damage to local vessels. Waves from the northeast, east, and southeast entered the harbor unobstructed, sometimes resulting in significant damage to vessels, operations, and facilities. During storm events, waves caused the vessels to impact pilings, causing various levels of damage, and sometimes resulting in sinking of a vessel. In addition, storm waves resulted in delays to navigation during attempts to dock a vessel, and were estimated to affect watermen approximately 75 days per year. Storm waves in Tedious Creek also had an economic impact on the crab shedding industry.

Several plans of improvement were examined at various structure heights for providing protection at Tedious Creek. Various structural and nonstructural solutions were evaluated, including breakwaters, bulkheads, and vessel relocation. Various breakwater layouts were selected for further detailed evaluation. The plan selected for construction was a breakwater in two sections each 245 m (800 ft) long, with one section connected to the north shore and one section connected to the south shore. Each section had a 15-m (50-ft) gap about midpoint location of both sections to improve water circulation in the harbor. There would be a central navigation gap 91 m (300 ft) wide located between the north and south sections of the breakwater.

The breakwater constructed in 1997 differed in geometry from the plans tested in 1994. Foundation problems encountered in the field resulted in a

shortening of the breakwater by about 30.5 m (100 ft) and, therefore, a wider opening between the two breakwater sections. As a result, it was anticipated that the level of wave protection provided would be different from the simulated plan. The same is true for tidal circulation impacts and their inferred sedimentation patterns.

After completion of the Tedious Creek breakwater project, local watermen complained that the breakwater was not providing the authorized level of protection at the county boat dock and public piers on the south shore. It was suggested that the as-built 122-m (400-ft) gap opening should be reduced to the authorized 91-m (300-ft) gap opening. The Baltimore District requested that Offshore and Coastal Technologies Incorporated—East Coast develop, calibrate, and verify a wave model for that area. Three different breakwater gap scenarios were modeled: (a) the authorized 91-m (300-ft) gap, (b) the as-built 122-m (400-ft) gap, and (c) a theoretical 61-m (200-ft) gap.

At about this time, the Baltimore District nominated the Tedious Creek breakwater project for monitoring and evaluation as part of the MCNP program. This nomination was subsequently authorized for funding by HQUSACE, and this MCNP study was conducted during the time period 2001 through 2004.

MCNP monitoring plan

The objective of the monitoring program was to determine if the harbor and its structures were performing (both functionally and structurally) as predicted by model studies used in the project design. Wave, current, sediment, and bathymetry measurements at the project site would determine the effectiveness of the functional design aspects. The structural aspects would be investigated using ground-based surveys and airborne photogrammetry.

Bathymetry data

Bathymetry data would be collected for the entire wetted area inside and just outside of the Tedious Creek area. The purpose was to provide an accurate baseline of bathymetric conditions now, and then again in the future, to determine the effects of the breakwater on bathymetry. Monitoring marsh formation within the protected harbor was also possible.

Ground-based survey

A limited ground-based survey would be conducted to establish control (monuments and targets) for photogrammetric analysis of the structures. Also, a walking inspection would be conducted to document any broken armor units or dislodged stones.

Photogrammetry

Changes that might be expected in Tedious Creek included movement of the breakwater, shoreline/wetland migration, and sediment redistribution. Methods used historically to document breakwater movement have used stereoscopic imagery to determine base versus plan differences in elevation. In cases where the stone size is conservatively large for the expected waves (as in Tedious Creek), it is possible to use recent advances in surveying methods, specifically the DGPS, to set up numerous monuments on individual units in the breakwater to document three-dimensional movements. The expected accuracy of such methods (<0.03 m (0.1 ft) vertically) is arguably greater than even the best stereoscopic analysis. Both methods would be used in the initial year. In the second year, breakwater movement would be detemined using (a) surveying and (b) digital photographic methods for the purpose of selecting the methods to be used in the final year. Both methods might be used in the final year.

Wave data

An ongoing Tedious Creek wave collection effort by the Baltimore District would be used to provide the data needed to evaluate the performance of the breakwater in attenuating waves. This effort included one-directional wave gage and two nondirectional wave gages deployed in a 3- to 4-month wave intensive season. The directional wave gage is located outside the breakwater, and the nondirectional wave gages are located just inside the breakwater opening and at the county boat dock and public piers on the southern side of the estuary. The wave observations would be correlated with numerical model results to determine the accuracy of the models as a design tool for such projects.

Tide data

Continuous recording tide gages would be located both inside and outside of Tedious Creek. The outside gage would be used to define a source tide that would be used to drive revalidation simulations of the tidal circulation

models. The inside gage would be used as a validation gage. One-week data collection exercises were planned in each of the first 3 years.

Current data

Current velocity data would be collected using overboard ADCP meters, and other methods as necessary around the breakwaters, to provide a data set with which the tidal circulation models can be validated. One-week data collection exercises were planned for the first 3 years.

Sediment Data

Bottom grab samples of sediment would be collected throughout the Tedious Creek area to determine sedimentation patterns. The locations would be biased to areas predicted by the circulation models to be areas of anticipated scour or deposition. Coarser materials are expected to occur in areas of higher velocities such as gaps in the breakwaters. Finer materials are expected in lower velocity areas near the breakwaters sheltered from tidal currents. Grain-size analyses would be performed and ultimately used in the numerical models to determine their ability to predict such processes.

Conclusions

Offshore and Coastal Technologies (2001) study

Numerical wave models STWAVE (no-diffraction) and STB3 (diffraction) were configured to perform wave transformation simulations within the harbor and at the county boat dock facilities on the south side of the estuary where most damages were occurring. The worst wave conditions at the boat dock appear to result from northeasterly offshore waves. Three scenarios were evaluated:

- Preproject conditions (no breakwater).
- Existing as-built conditions (122-m (400-ft) breakwater gap opening).
- Authorized project (91-m (300-ft) gap opening).

Wave height transformations were performed with varying wave heights, tides, storm surge levels, and incident wave angles. The simulations concentrated on conditions occurring at the county boat dock where fishing operations are affected by waves reported to be entering the harbor through the jetty gap.

Storm waves, as-built 122-m (400-ft) gap and authorized 91-m (300-ft) gap

During extreme storm wave events (100-year wave and water levels), the existing as-built breakwater with a 122-m (400-ft) gap reduces wave heights at the county boat dock by as much as 70 percent, as compared to the no-breakwater project conditions, from 2 m (6.4 ft) offshore to 0.5 m (1.7 ft) at the county boat dock. Narrowing the as-built breakwater gap width to the authorized project breakwater gap of 91 m (300 ft) resulted in very little difference in the storm wave heights at the county boat dock.

Moderate waves, as-built 122-m (400-ft) gap and authorized 91-m (300-ft) gap

During moderate wave conditions (0.9- to 1.2-m (3- to 4-ft) offshore wave heights), the as-built breakwater reduces wave heights at the county boat dock by as much as 50 percent at high tide and 30 percent at low tide, as compared to the no-breakwater conditions, to a wave height of 0.1 m (0.3 ft) at low tide and 0.2 m (0.6 ft) at high tide. The authorized project with a gap width of 91 m (300 ft) did not result in any difference in the wave heights at the county boat dock for either low or high tides.

Typical waves, as-built 122-m (400-ft) gap and authorized 91-m (300-ft) gap

During typical daily conditions (0.3- to 0.6-m (1- to 2-ft) offshore wave heights), neither the as-built nor authorized projects result in any reduction in wave heights at the county boat dock at a mid-tide level of 0.5 m (1.6 ft). Wave heights at the county boat dock, however, are transformed by the natural bathymetry of the creek to a tolerable level of less than 0.2 m (0.5 ft) for no-breakwater conditions, and for all breakwater gap widths.

Storm and typical waves, hypothetical 61-m (200-ft) gap

Offshore and Coastal Technologies also modeled a hypothetical 61-m (200-ft) gap width. This scenario would require modification of the authorized project to extend the breakwaters an additional 61 m (200 ft) beyond the as-built project or 30.5 m (100 ft) beyond the authorized project. The models STWAVE and STB3 demonstrated that narrowing the breakwater gap width to 61 m (200 ft) would result in an insignificant difference in the storm wave heights at the county boat dock, and a 10-percent reduction in wave heights during typical daily conditions. However, since the as-built project was shown to reduce normal daily wave

heights at the county boat dock to less than 0.2 m (0.5 ft), which is considered a tolerable level for vessel unloading and mooring, an additional 10-percent reduction would be considered insignificant and modification of the project would not be justified.

Locally generated waves on Tedious Creek estuary

Field data and observations made by the Baltimore District during project location site visits indicate that wave conditions preventing satisfactory operations at the county boat dock facility often result from northwesterly waves generated locally on Tedious Creek, rather than by waves propagating through the breakwater gap from a northeasterly direction.

MCNP field data collection and analysis

High-quality field data measurements are an integral part of the design process for either new projects or existing engineering project modifications. Seven different types of field data collection were conducted over a 2-year period: (a) positioning and datum referencing, (b) aerial photography, (c) tidal data collection, (d) wave data collection, (e) surveys (hydrographic, bank line, and breakwater), (f) bottom sediment sample collection, and (g) current velocities. Dates of the field data collections were August 2001, September 2002, March 2003, May 2003, and August 2003.

Positioning and data referencing

Construction benchmarks near the jetty structure were surveyed to NOS tidal benchmarks to check the validity of the construction benchmarks and to determine the appropriate offsets for locating the GPS base station onsite while conducting all survey operations. These offsets will also provide a check in future surveys in the area to monitor subsidence at the site since the first order points are driven to a point of refusal.

Aerial photography

Targets are first priority in obtaining precision aerial photography. Aerial targets were painted on road intersections around the project using white paint outlined in black. Ideally, a point in each corner of the image gives excellent results. This aerial photographic data were acquired to define the wetted perimeter at an instant in time, and to monitor the shoreline degradation by comparing surveys from future years. If the shoreline

position changed significantly between subsequent surveys, then the erosion or deposition rates could be estimated. Since meter-level accuracy was sufficient for the numerical modeling requirements at this project location, standard aerial photographic techniques were employed.

Tidal data collection

Water-level data were acquired by pressure tide gages deployed at three locations during the field studies in August 2001 and August 2003. These tide gages were programmed to collect changes in water elevation every 15 min. During processing, the established elevation and the change in the water surface were used to process the hydrographic data. These data were used to produce an accurate graph of the tidal cycle, and for the RMA2 numerical simulation model simulations. These measurements provide short-term records of a few days for comparing numerical model results, and for correcting bathymetric survey data. In addition to these short-term data, water surface elevations were recorded for several weeks at a time.

Wave data collection

Three wave gages were deployed on 12 March 2003 and recovered on 20 May 2003. Two gages inside the jetties were nondirectional gages attached to pilings that recorded 2,048 measurements of pressure at a 2-Hz rate every 2 hr. The pressure measurements were converted to surface elevations when the data were processed. The wave gage outside the jetty was a bottom-mounted Sontek Acoustic Doppler Velocimeter with a pressure sensor. Directional wave data are obtained from this instrument from nearly collocated measurements of wave-induced velocity and pressure. The wave heights were obtained from the pressure measurements, after converting them to surface elevations during data processing, and were used for CGWAVE calibration and verification.

Hydrographic surveys

A survey boat was equipped with GPS (for positioning), an Odom Hydrotrac Fathometer (for depth), and a laptop computer using HyPACK (hydrographic data collection software). The hydro-survey crew collected data along lines that had predetermined positions. The lines were drawn digitally, in the survey program, onto maps provided by the Baltimore District. The hydro-survey crew piloted the boat along each line collecting both position and depth data simultaneously. A second bathymetric survey

was conducted in August 2003 to ascertain any changes that may have occurred in this region since the original bathymetric survey that had been conducted in August 2001. There were no detectable significant differences between the results of the two surveys.

Bank line surveys

The survey vessel was used to transport personnel around the perimeter of the Tedious Creek area. A member of the field crew stood on the front of the boat with the RTK-GPS unit and, as the boat pulled into the bank, would collect a data point at the water line. These bank line data were collected approximately every 15.2 m (50 ft) around the perimeter of the bay. Bank line surveys were conducted in August 2001 and August 2003. There were no statistically significant changes in the bank line elevations between the two surveys.

Breakwater surveys

The ability to return to the breakwater at a later date and survey the same point on each stone was important. This would be possible by using a GPS program and a handheld computer. Data were collected along the entire length of the breakwater with cross sections every 6.1 m (20 ft). The breakwater surveys conducted in August 2001 and August 2003 measured elevations along cross sections across the breakwater. The surveys were conducted to determine if there had been any changes in the elevations of the breakwater sections during the interval between the two surveys. For all the cross sections, the average change in elevation between the two surveys ranged from 0.04 to 0.09 m (0.13 to 0.28 ft), with a maximum of 0.20 m (0.67 ft). These are distances in a vertical downward direction, indicating minimal and insignificant settling of breakwater stones.

Bottom sediment sample collection

Sixty-four bottom sediment samples were collected using a clamshell sampler throughout the Tedious Creek estuary. The samples were stored in 4.4-L (1-gal) ziplock freezer bags for shipment back to the laboratory for grain-size analysis. Standard sieve analysis was performed on these samples. The sample locations were scattered throughout the project area to define the spatial variability of sediment types. The sediment data were imported into the HyPAS GIS system for analysis and plotting. The sediment toolbox was used to generate gradation curves. This toolbox

allows the user to look at all the samples in relation to other data types (bathymetry, velocity, and aerial photography).

Current velocities

The ADCP survey involved running three separate lines in the area of interest. The ADCP current transect surveys were conducted in August 2001, September 2002, and August 2003. Each line was run one time each hour for the 13-hr time period. The purpose of this effort was to capture the flow conditions throughout a tidal cycle at critical points in the system. Numerical modelers require the total discharge into the system from Tedious Creek and Fishing Bay. The centerline was located so as to capture eddies generated from the flood jet as it entered the small bay. All of the velocity data for each of the 13-hr ADCP data sets were imported into HyPAS for plotting displays, and used for comparisons with the RMA2 numerical model results. RMA2 is a two-dimensional, depth-averaged finite element hydrodynamic model that computes water surface elevations and horizontal velocity components for subcritical, free-surface flow.

CGWAVE numerical model comparisons between existing as-built and authorized breakwater configurations

CGWAVE setup and calibration

CGWAVE is a general purpose, state-of-the-art wave prediction model based on the mild slope equation that is used to model waves in harbors, open coasts, inlets, and around islands and fixed and floating structures. It includes (a) effects of wave refraction and diffraction, (b) dissipation from bottom friction, wave breaking, and nonlinear amplitude dispersion, and (c) harbor entrance losses. CGWAVE is a finite element model that is interfaced with the SMS for graphics and efficient implementation (pre- and post-processing). CGWAVE requires a minimum of 6 to 10 elements per wavelength. Since the harbor area is large and shallow and wave periods as small as 6 sec are prevalent, the required grid size was determined to be 1.9 m (6.1 ft). The model size was limited to cover only the most critical areas inside and outside the harbor. Because local interest was in the areas near the county boat dock and the public piers, two transects were selected between these two facilities and the breakwater entrance. Computational boxes were established around the county boat dock and the

public piers. CGWAVE was calibrated with field data acquired between April and July 2001.

As-built breakwater configuration

For the existing configuration, larger wave heights occur to the north for waves traveling toward the northwest, to the west and vicinity of the public pier for waves traveling toward the west, and to the southwest and vicinity of the county boat dock for waves traveling toward the southwest. For both transects, the largest wave height was less than 0.78 m (2.6 ft) and the average wave height was less than 0.31 m (1.0 ft). The maximum wave heights in any box for any wave condition were less than 0.37 m (1.2 ft) and 0.30 m (1.0 ft) for the public pier and county boat dock computational boxes, respectively. Finally, the 95-percent confidence intervals for average wave height inside all boxes for all wave conditions were 0.16 ± 0.02 m (0.52 ± 0.07 ft) in the public pier area and 0.16 ± 0.03 m (0.52 ± 0.10 ft) in the county boat dock area.

Authorized breakwater configuration

For the authorized configuration, the narrower gap reduces waves traveling to the southwest more than to the west along both transects. The difference in wave height for waves traveling to the west and waves traveling to the southwest decreases as wave period increases, however. The effect of the smaller gap was minimal on overall wave height reduction, in agreement with the Offshore and Coastal Technologies Incorporated (2001) results. The largest wave height along the two transects was 0.86 m (2.8 ft), with an average slightly less than the existing configuration. The difference in maximum wave heights is probably due to reflections at the breakwater entrance from the longer breakwater in the authorized configuration. The maximum wave heights in any of the boxes for any wave conditions in the public pier and the county boat dock areas were 0.38 m (1.3 ft) and 0.23 m (0.8 ft), respectively. Finally, the 95-percent confidence interval for average wave height inside all boxes for all wave conditions was 0.13 ± 0.02 m (0.43 ± 0.07 ft) in both the public pier and the county boat dock areas.

Comparison between as-built and authorized breakwater configurations

Wave height differences were calculated between the wave heights predicted for the existing 122-m (400-ft) gap and the authorized 91-m

(300-ft) gap. Because of the variability along the two transects, only the wave heights in the boxes were compared. In some cases, the existing configuration had lower wave heights. Only the positive differences were reported here as these represent cases where the authorized configuration would have resulted in smaller waves inside the harbor. In general, the maximum difference wave heights are not very large and occurred for southwest waves for both the public pier and the county boat dock boxes. The largest wave height differences were less than 0.09 m (0.3 ft) and 0.21 m (0.7 ft) for the public pier and the county boat dock boxes, respectively. Finally, the 95-percent confidence interval for the average difference wave height inside all boxes for all wave conditions was 0.03 ± 0.01 m (0.10 ± 0.03 ft). Any wave height reduction that would have been afforded by the smaller entrance gap had the authorized configuration been constructed is truly insignificant.

Comparison between CGWAVE and Coastal and Offshore Technologies (2001) study

Comparisons with the Offshore and Coastal Technologies Incorporated (2001) STWAVE numerical model study are in general agreement with the wave heights and directions predicted by their model.

RMA2 hydrodynamic verification and RMA4 flushing analysis of Tedious Creek existing as-built condition

RMA2 two-dimensional hydrodynamic model verification to August 2001 event

RMA2 solves the two-dimensional, depth-averaged equations governing shallow water. It uses the Reynolds form of the nonlinear Navier-Stokes equations, and includes phenomenological terms such as bed shear stress, wind stress, wave stress, and coriolis effects. RMA2 is a finite element model for subcritical open-channel flow. It computes velocities, flow rates, and water surface elevations in rivers, estuaries, wetlands, etc. Two of RMA2's more popular features are the automatic parameter selection and the model's ability to handle wetting and drying. RMA2 was verified by (a) water surface elevations, (b) velocities, and (c) general circulation patterns.

For the water surface elevations verification, the two field tide gages (TG-346 and TG-347) that fall within the numerical model domain were

compared to the solution obtained by the RMA2 numerical model. There is excellent agreement in the comparisons, which is demonstrated both graphically and statistically.

For the velocities verification, there was no stationary point velocity gage. Instead, the survey boat ran survey lines at 1-hr intervals with the ADCP equipment and measured the water speed and direction for numerous water depth profiles. Representative snapshots of that data were chosen to represent each maximum flood and ebb event around the 400-ft-wide gap opening, and were reduced in HyPAS to represent a “depth-averaged” condition. This reduction was required in order to accurately compare these to RMA2, which is a depth-averaged solution. Unlike the statistical measures, this type of comparison is “subjective.” The agreement appears very good for the maximum flood vectors. However, there is slight disagreement in the vicinity of the northern gap for the maximum ebb eddy pattern.

For the general circulation patterns verification, SMS has the ability to post-process vector patterns, flow traces, and drogue plots for either a steady-state event or for a transient simulation of the RMA2 solution. These tools can give insight to understanding the general circulation patterns and flushing characteristics of the harbor. The most direct method of understanding the circulation pattern is with an SMS velocity vector plot, again concentrating on the portion of the simulation where maximum ebb and maximum flood currents occur. The drogue plot capability within SMS allows a set of “drogue points” to be placed within the computational domain and allowed to “float” with the currents. A representative set of drogues was placed within the numerical solution and carried with the currents for as much as 70 hr. Another method to examine general circulation patterns in a tidal environment is to analyze the “net velocity” over one or more complete tidal cycles. Both methods (drogue plots and net velocity) showed very good results, with only the small northern gap showing a slight flood residual dominance.

RMA4 two-dimensional water quality constituent transport model

RMA4 was used to quantify the extent to which the as-built existing condition harbor has the ability to flush. RMA4 is designed to simulate the depth-averaged advection-diffusion process in an aquatic environment. The model can be used for the evaluation of any conservative substance

that is either dissolved in the water or that may be assumed to be neutrally buoyant within the water column.

The goal of the RMA4 flushing simulations was to conduct experiments to determine if the Tedious Creek Harbor flushed a contaminant within a reasonable period of time. The first of these experiments initializes the entire numerical domain to a zero concentration, and then drops a conservative contaminant near the northern edge of the boat dock. The contaminant would issue its load for 4 hr, and then be turned off. The incoming tidal waters would be tagged as fresh water. The flood/ebb hydrodynamic cycle would be repeated until the flushing test concluded.

RMA4 Flushing Demonstration No. 1 was initialized to 0.0 ppt everywhere. A small element situated near the tip of the boat dock was selected as the load location. The simulation demonstrated that the contaminant spilled near the Tedious Creek boat dock flushed within 107.5 hr. This simulation was re-run using the no-structures hydrodynamic solution from RMA2. The results without structures showed that the 4-hr spill at the dock was cleaned to 0.01 ppt within 131.0 hr as compared to 107.5 hr for the same flushing demonstration with as-built structures.

RMA4 Flushing Demonstration No. 2 was initialized to 1.0 ppt everywhere except east of the structures, which was initialized as fresh (0.0 ppt). The simulation ran 5,760 transient 15-min time steps for a total of 1,440 hr (60 days). At the conclusion of this simulation, there was still some contaminant in the 0.2-ppt range in the upper tidal creek. In fact, little cleaning of this tidal creek is evident beyond time = 900 hr.

Conclusions

The as-built structures appear to maintain good harbor circulation, with velocities below any threat to boats that frequent the harbor. Additionally, the RMA4 flushing tests indicate that the Tedious Creek Harbor has adequate flushing and compares favorably to the no-structure flushing test.

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Appendix A
How to Use CGWAVE with SMS: An Example for Tedious Creek Small Craft Harbor[1]

Purpose

This appendix presents an example of how to use the numerical model Coastal Gravity Wave (CGWAVE) within the Surface Water Modeling System (SMS) Version 8.0 environment. Updates of SMS are frequent and may result in different control options. This appendix was completed as part of the Tedious Creek, MD, work unit of the Monitoring Completed Navigation Projects program.

Background

SMS is a comprehensive graphical user interface for model conceptualization, mesh generation, statistical interpretation, and visual examination of surface-water model simulation results. The version described herein is SMS Version 8.1. It is the main model delivery system with pre- and post-processor capabilities for all the U.S. Army Engineer Research and Development Center (ERDC) Coastal and Hydraulics Laboratory numerical models, including ADCIRC, TABS (RMA2, RMA4, SED2D), ADH, HiVEL, M2D, STWAVE, BOUSS2D, and CGWAVE. These integrated or interconnected models provide circulation and/or wave climate from a range of coastal processes including waves and currents, sediment transport and morphology change, channel infilling and inlet morphology, and dredged material fate. SMS provides the tools and macros for editing and display for mesh development, coordinate conversion, model connectivity, animations, and comparisons.

SMS is divided into modules. The modules discussed herein are Scatter, Map, and Mesh. The Scatter module stores scattered data sets, such as bathymetric data, and interpolates them to model grids and meshes. The Map module is used to create and manipulate conceptual models. The user can create and define attributes for feature objects, such as points, arcs, and polygons, that define the system being modeled. Images such as

[1] This appendix is extracted essentially verbatim from Briggs et al. (2004).

scanned maps or photographs can be helpful in this process and also make the display easier to interpret. Drawing objects such as text, lines, and arrows further improve the readability of the display. The Mesh module and other modules are used to create and manipulate numerical models. They allow (a) graphical interaction with mesh/grid editing, boundary conditions, model parameters, and materials, and (b) visualization of numerical model layout, and solution data. Each of the modules used to create and manipulate numerical models may have different coverage associations. For instance, the Mesh module has many coverages, one for each model supported.

CGWAVE capabilities

CGWAVE is a general purpose, state-of-the-art finite element wave prediction model based on the elliptic mild-slope wave equation that is applicable to harbors, open coasts, inlets, islands, and fixed and floating structures. It includes the effects of wave refraction, diffraction, and dissipation from bottom friction, wave breaking, nonlinear amplitude dispersion, and harbor entrance losses. Wave breaking can also be added during post-processing if not activated during model execution. Bottom friction is especially important for long waves. CGWAVE does not have any wind input or wave runup/overtopping of structures. Wave-current and wave-wave interaction processes are in development.

CGWAVE accepts arbitrary domains (Figure A1) and structures using linear triangular finite elements. The large number of discretized equations is solved with iterative and direct solvers as a steady-state problem. Convergence is guaranteed, but can be extremely slow on PCs with large model domains. The ERDC High Performance Computing Center's (HPC's) supercomputers can be used to quickly solve large problems. Currently, the Silicon Graphics, Inc. (SGI), solver available on the SGI Origin 3000 supercomputer named "Ruby" provides reasonably fast run times with CGWAVE. The model output is then transferred back to the PC for post-processing.

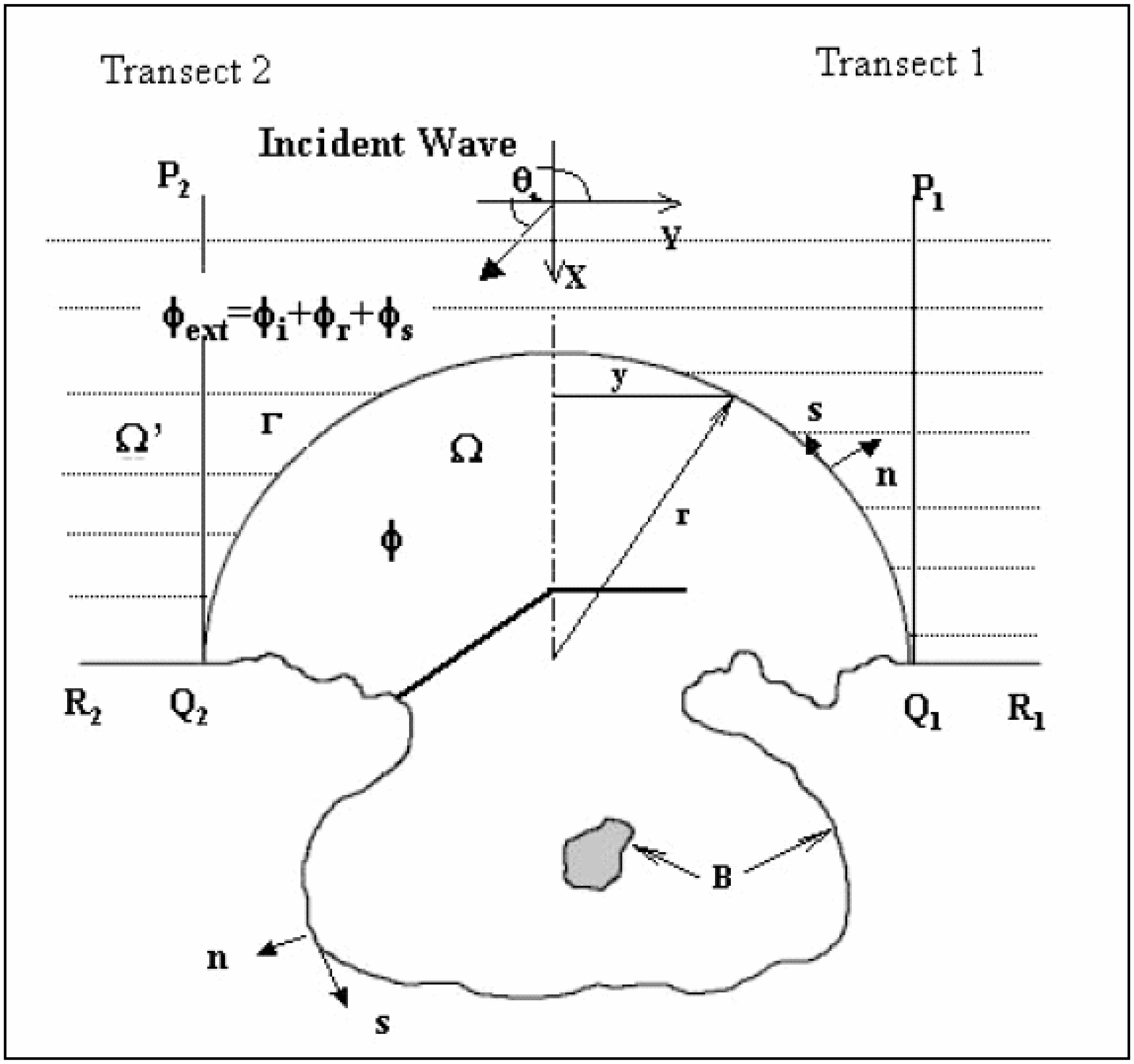


Figure A1. Example of a CGWAVE model domain.

Both regular and spectral waves can be input. Spectral waves (irregular waves) can be simulated by combining regular wave cases. The wave direction is the direction the waves travel to and is measured positive in a counterclockwise direction from east (i.e., 0 deg). Short and long waves, including tsunamis, can be modeled.

Additional information on CGWAVE can be obtained from Demirbilek and Panchang (1998), and from *http://chl.wes.army.mil/research/wave/wavesprg/numeric/wentrances/cgwave.htp*.

STWAVE versus CGWAVE

What is the difference between these two numerical models? When do you use one instead of the other? Both are used for wave modeling, but the relative scales are different. Wave modeling can be grouped into three classes or relative scales: generation, transformation, and local scale. The generation scale deals with waves as they are formed or generated in deep water. The transformation scale describes wave transformation in deep to intermediate level water depths. The finite difference model Steady Wave (STWAVE) falls in this class of numerical models and includes the coastal processes of refraction, shoaling, wave-current interaction, wave growth, and depth-limited wave breaking. The local scale class is for nearshore or local modeling of intermediate to shallow water. The finite element CGWAVE model is in this category. Thus, the STWAVE can be used to transform deepwater waves to a depth coinciding with the offshore boundary of the CGWAVE model.

CGWAVE procedure

The procedure for running CGWAVE within SMS can be divided into four parts:

- Create a conceptual model.
- Generate the finite element mesh.
- Run the model.
- Post-process the results.

Each of these steps is discussed in the following paragraphs. Since SMS does not have an "undo" function, it is a good idea to save your work early and often.

Create conceptual model

To create the conceptual model that SMS uses to generate a mesh-based numeric model, you should (a) import and register a background image, (b) gather coastline and bathymetric data, (c) determine the coordinate system and datum reference for the project, and (d) conceptualize the study area with a CGWAVE coverage type.

Background image

It is useful to have an aerial photograph or image of the project area to see if the conceptual model is correct. Image data is input in the Map module in .tif or .jpg formats using the **File|Open** command. These images have to be registered or geo-referenced to state plane coordinates, meters. The three registration points shown in Figure A2 must be moved to known coordinate positions or a pre-existing geographic information system type “world file” must be read into SMS to geo-reference the image. In this example, the *.jpg file was opened first and then the .jgw world file was input using the **Import World File** button. If an SMS image file (.img) was previously saved, it can be opened in lieu of the world file referencing. The ASCII image file tc_overlay_metric.img opens the file Tedious_small.jpg and provides the proper registration information.

Coastline and bathymetric data

The CGWAVE model requires bathymetric and coastline data. Bathymetric data is input in the Scatter module in an “xyz” file format, using the **File|Open** command. The x- and y-coordinates should be in state plane coordinates referenced to true north. All values of water depth should be in meters with positive values being downward. Bathymetry should include the model domain and a sufficient distance offshore to cover the one-dimensional (1D) transects (see following section). Sources of bathymetric data are previous studies, local surveys, and databases including GEODAS, the worldwide geophysics database. In this study, survey data from August 2001 were available in the file Tedious_Aug01_harbor_water_depth_meter.xyz for the inner harbor and Ted_cr_mllw_m.xyz for the offshore area. The bathymetric data are named elevation, by default.

Coastline data define the “wet” edge of the model for the land-water interface. The data file Tedious_Aug01_landedge_survey_meter.xyz was obtained by walking the bank line with a hand-held global positioning system and was used to define the land edge. It was also input in the Scatter module. Figure A3 shows the bathymetry and coastline scatter sets overlaid on the background image.

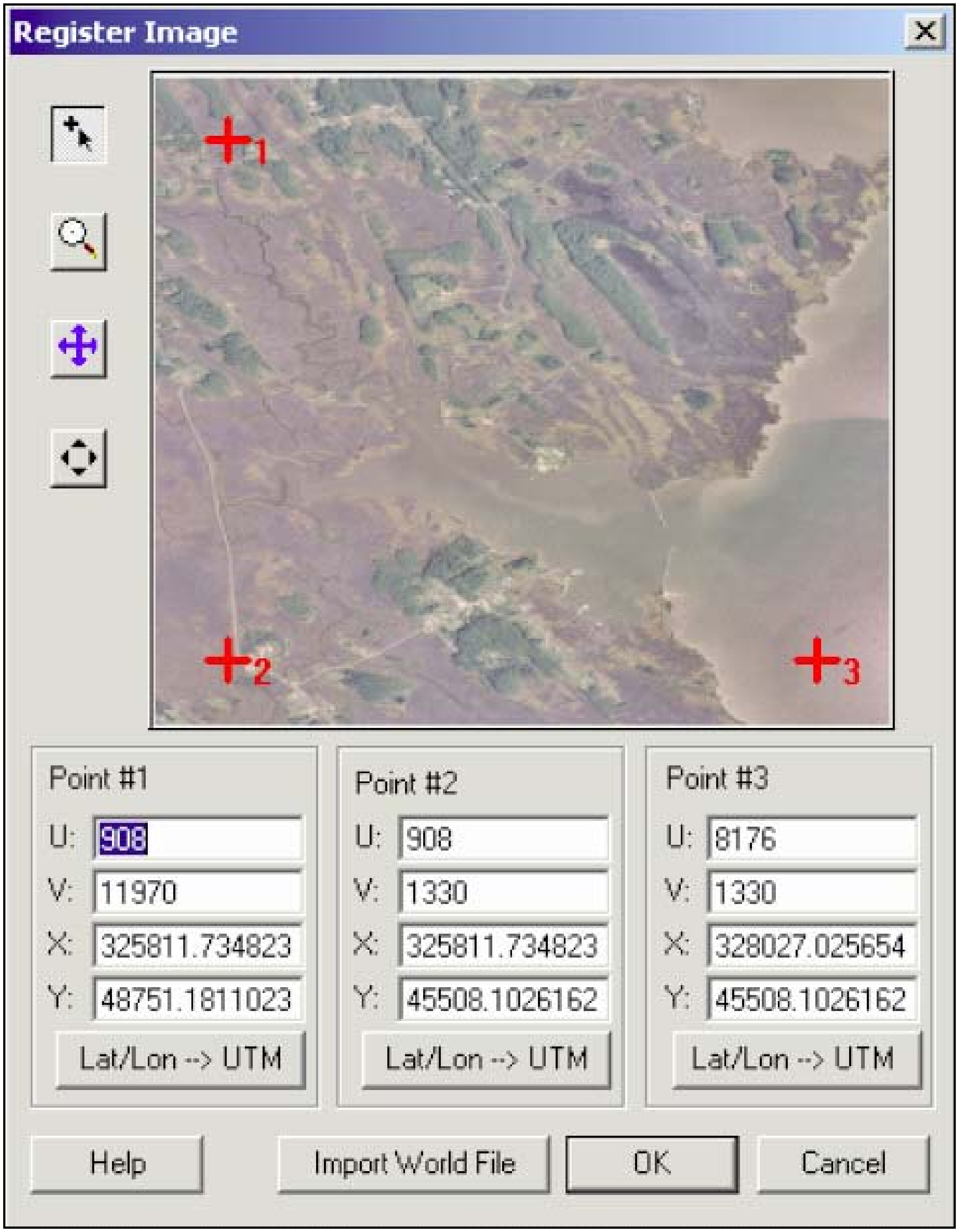


Figure A2. Registering image of Tedious Creek, MD.

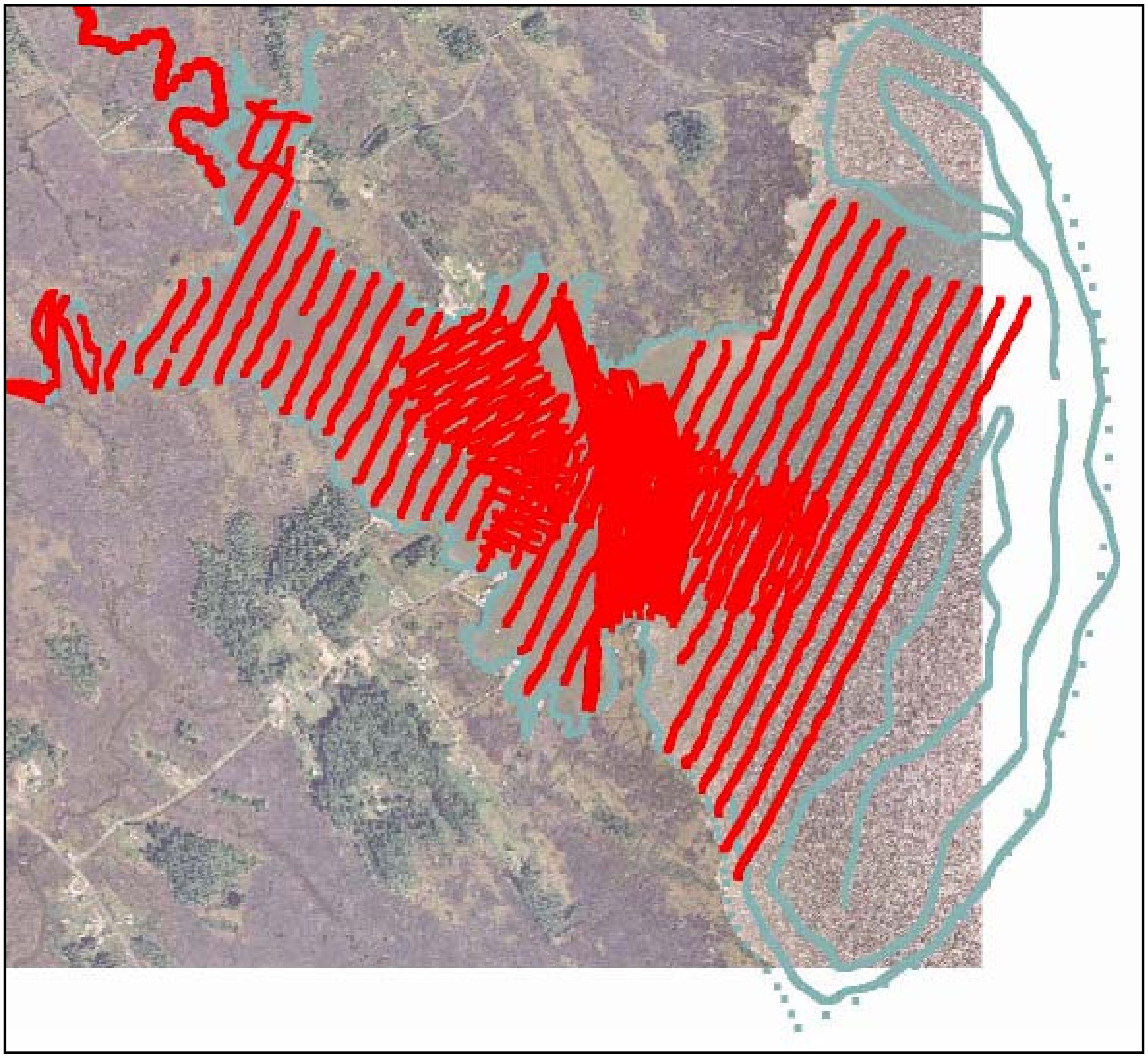


Figure A3. Scatter data points representing bathymetric survey locations for Tedious Creek.

Coordinate system and datum reference

All data must be geo-referenced to be useful. So the bathymetric and coastline data must also be converted to the same coordinate system to be properly merged. A copy of the "*CORPSCON*" conversion utility is contained within SMS. Since the earth is round, it is not possible to have exact x/y-coordinates based on latitude and longitude. Thus, the world is divided into smaller rectangular sections that permit more precise measurement in horizontal and vertical directions. The University of Texas Web site *http://www.ncgia.ucsb.edu/education/cirricula/giscc/units/u013/u013.html* gives a good description of the different coordinate systems.

Horizontal coordinate systems supported within SMS are (a) geographic, (b) State Plane, (c) Universal Transverse Mercator (UTM), and (d) NAD 27/83/HPGN coordinates in meters, U.S. survey feet, or international feet. Geographic coordinate systems consist of latitude, longitude, and height measurements, with the prime meridian and the equator as reference planes to define latitude and longitude. The State Plane System was developed in the United States in the 1930s to provide a national datum and was based on the North American Datum 1927 (NAD 27), with coordinates in feet. In 1983 the NAD 27 was superseded by the NAD 83 system with units in meters. Smaller states may have only one zone, where larger states may be divided into several zones. UTM coordinates are composed of UTM zone numbers designating 6-deg longitudinal strips and UTM zone characters designating 8-deg latitude zones. Vertical coordinate systems support National Geodetic Vertical Datum (NGVD) 29 or North American Vertical Datum (NAVD) 88 in meters, U.S. survey feet, or international feet. The mean low low water datum is usually used. These conversions also make sure all data is aligned according to true north.

Figure A4 is an example of a single point conversion within SMS for the offshore directional NORTEK Aquadopp wave gage used in the field measurements at Tedious Creek during April to July 2001. Water depth was 2.3 m (7.5 ft). The gage was located at lat. 38.24482° and long. 76.03995°. Click on the **Edit|Single Point Conversions** command to open the conversion routine. Values can be input in decimal degrees or in degrees, minutes, and seconds. The input and output horizontal systems are geographic NAD 27 (US) and State Plane NAD 27 (US), respectively. The output State Plane Zone is Maryland – 1900. These can be determined using the *Help* menu and locating the particular site on the maps. The vertical system coordinates are not used here, but still must be input correctly. The input and output vertical systems are local and NGVD 29 (US), respectively, with a value of 7.5 U.S. survey feet. Clicking the **Convert** button results in an x value of 327884.3540 m, y of 46117.0956 m, and z of 2.2860 m.

In summary for the Tedious Creek project, all coordinate systems were converted to meters. The horizontal coordinate system is the State Plane “NAD27 Maryland – 1900,” and the vertical datum reference is NGVD 29 (US).

Conceptualize the study area

The conceptual model should define (a) the model domain, (b) feature arcs for the coastline boundary, open ocean boundary, and any additional structures, and (c) the 1D transects. Before you begin to conceptualize the study area, create a CGWAVE coverage by selecting **Feature Objects|Coverages** in the Map module.

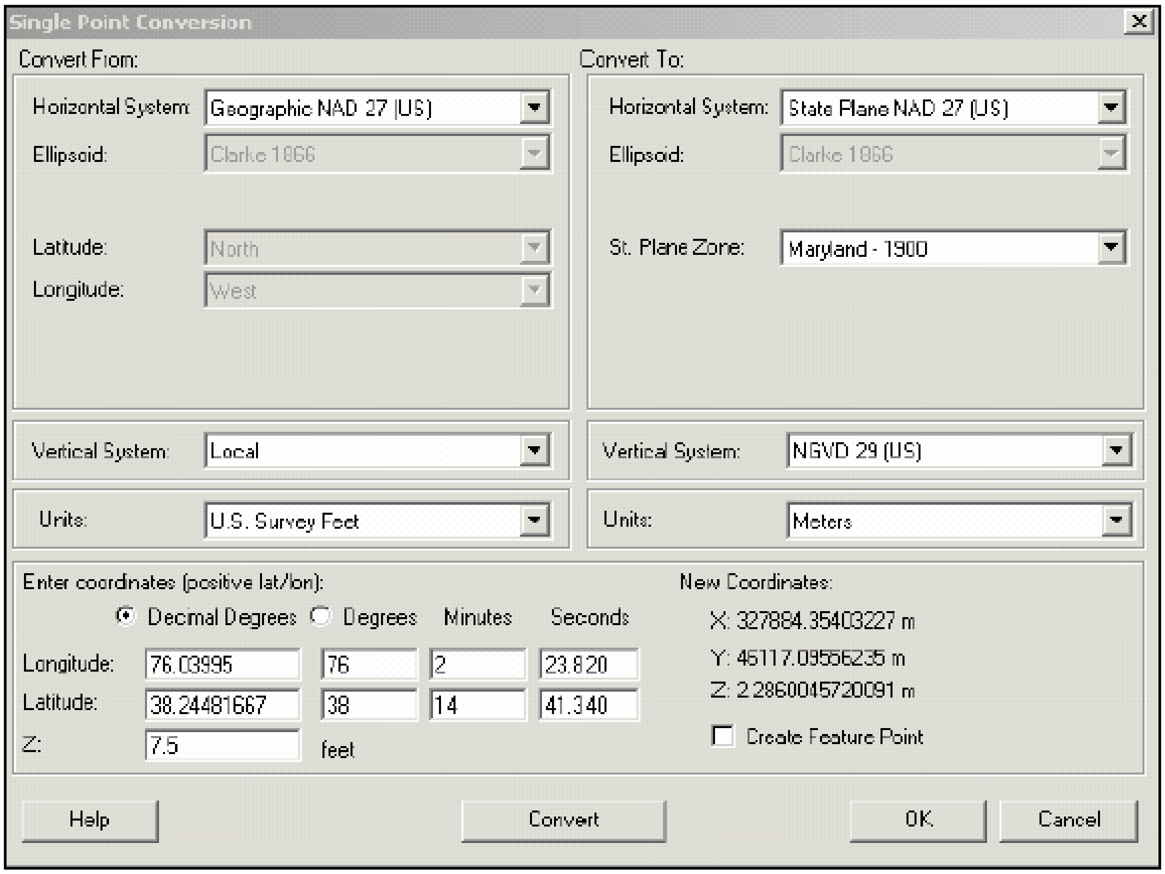


Figure A4. Single point coordinate conversion.

Model domain

The first step toward defining the numerical model domain is having a mindset of the study purpose and the computational capability (processor speed and memory) to solve the problem. The main items of interest should be within the inner one-third of the model domain. CGWAVE uses a size function, which takes into account the shortest expected wavelength of concern and the water depth to define the required resolution for accurate computation. Although this will be discussed in detail later, it is a good idea to do a quick estimate of the anticipated number of elements.

Figure A5 shows the model domain for the Tedious Creek model. The west or back end of the model was truncated in water within the harbor instead of the actual shoreline to reduce the number of nodes and elements required. Because it is marshy and shallow in the upper reaches of the harbor, it was not necessary to model these areas to achieve accurate results in the study area.

Feature arcs

Create the coastline arc by selecting **Feature Objects|Create Coastline**. SMS will automatically examine the bathymetry contours to create a feature arc at a user-specified elevation. Enter the minimum water depth to define the coastline or land-water interface. A value of 5 cm (i.e., 0.05 m) is usually selected as the coastline water depth to ensure that all elements are wet. Adjustments can be made in the coastline data file to make sure there are no zeros in the definition. The coastline arc is shown in brown in Figure A5.

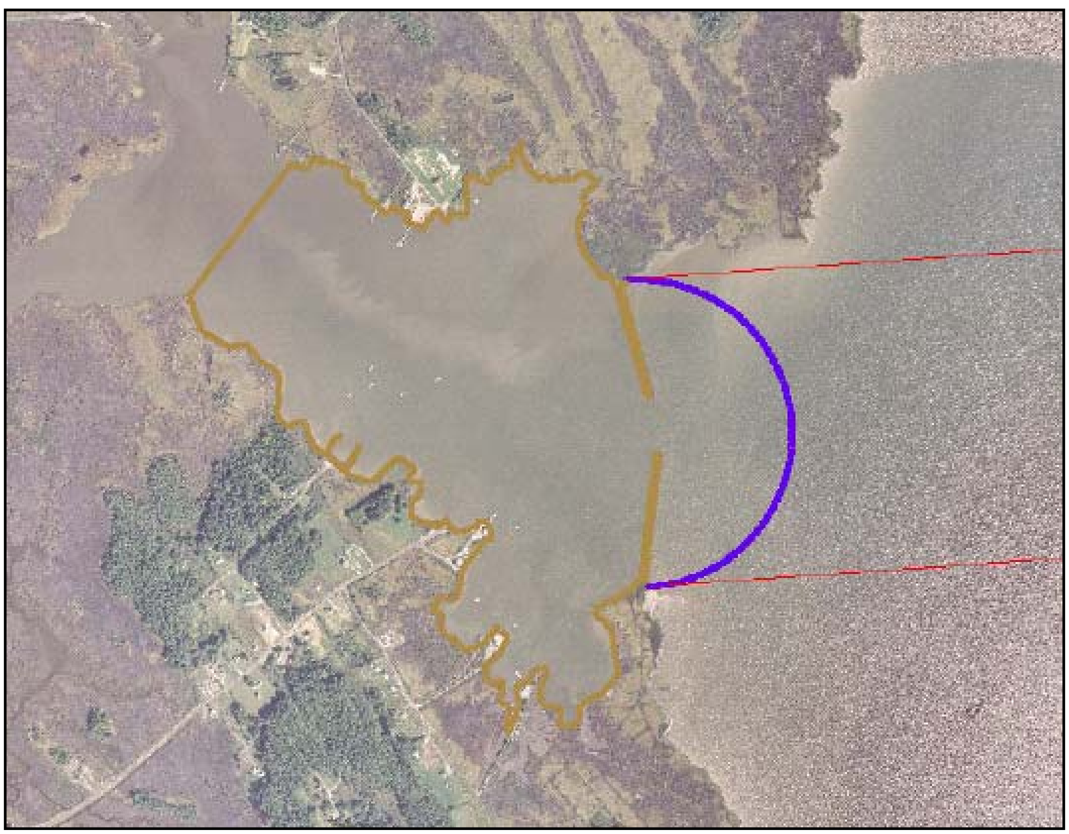


Figure A5. CGWAVE model boundary for Tedious Creek, MD.

The ocean boundary is semicircular for a coast, but can be circular if the domain is an island. The radius of the semicircular arc of the ocean boundary is selected to include a distance upcoast and downcoast and offshore to permit sufficient wave transformation. Choose the *Select Feature Vertex* tool and select two end point vertices for the ocean boundary intersection with the coastline arc. The points should be approximately equidistant from the center of the harbor. Be sure to hold the shift key down while selecting both vertices. The domain is defined in the **Feature Objects|Define Domain** command. Choose the semicircular option for a shoreline. If the arc is oriented landward instead of offshore, select the arc to reverse direction using the **Feature Objects|Reverse Arc Direction** command in the Map module. Since the arc direction is determined by which node of the arc is selected first, deleting the original arc and re-entering the two end points in reverse order can also reverse the ocean boundary. The semicircular open ocean arc is shown in blue in Figure A5.

You can further define the area being studied by creating additional feature arcs. This is done using the *Create Feature Arc* tool in the Map module. The Tedious Creek model has a feature arc at each major change in direction along the coastline boundary. In the vicinity of the public piers, there are many arcs to properly define the piers. The coastline arc near the piers was broken into several unique arcs and vertices were moved to follow the exact layout of the piers (see section on “Reflection Coefficients”).

1D transects

The 1D lines are used to transform offshore wave data to the ocean boundary of the model to increase the reliability of the model’s predictions in projects where exterior bathymetric effects might play an important role. In cases where 1D transformation is used, it will no longer be necessary to use other spectral wave models to transform waves from deep water. The CGWAVE model uses a 1D mild slope wave equation to solve for the wave transformation from an offshore point with known wave climate to the CGWAVE semicircular ocean boundary. The assumption is that the bathymetry only changes in the offshore direction seaward of the ocean boundary, thus simplifying the calculations.

A 1D transect extends offshore from each side of the model domain (Figures A1 and A5) with origin at the intersection of the semicircular

boundary and the coastline boundary. Since the ocean boundary is curved, the values of wave transformation are input from the transect that is closest to the point along the semicircular boundary. Near the center, the values are averaged between the two transects. Each transect includes bottom friction and reflection, if these processes are activated. It is important to activate these processes unless the water depth becomes deep close to shore (i.e., in essence deepwater waves at the semicircular boundary) or the depth is relatively constant all along the ocean boundary.

Generate finite element mesh

To generate a good mesh for CGWAVE, you should (a) define the mesh resolution, (b) calculate size function, (c) build polygons, (d) assign reflection coefficients, (e) create mesh, (f) check mesh quality, and (g) assign CGWAVE model parameters.

Define mesh resolution

The mesh in the CGWAVE model is wavelength-dependent. Since wavelength is a function of the water depth and wave period, it is important to define the design or smallest wave periods of interest to the project. A minimum of 6 to 10 elements in the finite element mesh per wavelength is required to properly define the domain. Fifteen elements per wavelength is ideal. For relatively shallow projects like Tedious Creek, this is critical as the mesh may require a large number of elements, severely taxing the model’s capabilities. Ten or more elements per wavelength gives the best resolution of the wavelengths.

Calculate size function

The size function is calculated in the Scatter module. This is a two-step process: create the wavelength function and scale wavelength to create the size function. The first step (Figure A6) is to define and create the wavelength by specifying the design wave period in the **Data|Create Data Set** command of the Scatter module. Turn off all options except for the *Transitional Wavelength and Celerity* and enter a value of 6 sec for the design wave period. This is the smallest wave period with significant energy and frequency of occurrence for the project site. Make sure that the water depth data set (i.e., elevation) containing the bathymetry data is active. Two new data sets, *Transition Wavelength* and *Transition Celerity*, are created. The data set name can be left with the default *Transition* or

other more descriptive name. The *Transition Celerity* data set is not needed and can be deleted in the *Data Browser* if desired.

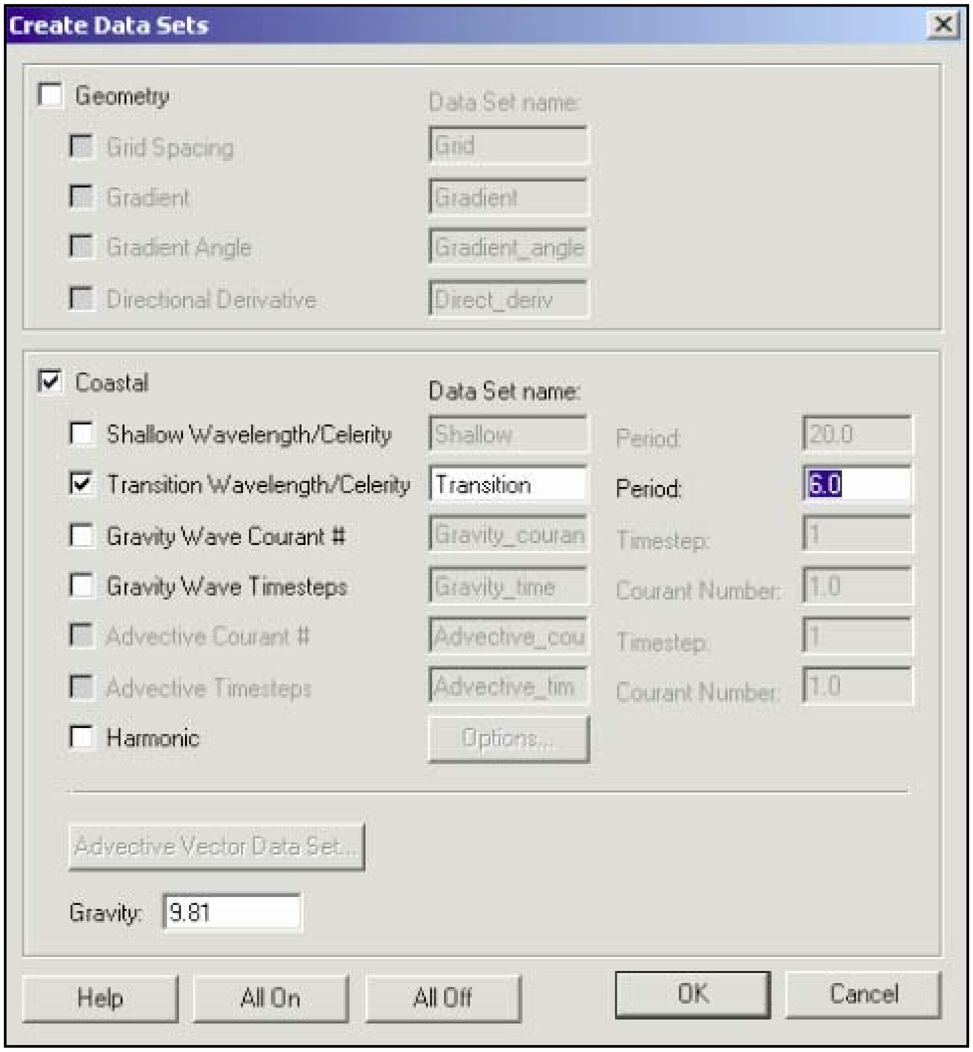


Figure A6. Creating a wavelength function for a wave period of 6 sec.

The next step (Figure A7) is to create a size function data set based on the *Transition Wavelength* in the **Data Calculator** command. Select the *Transition Wavelength*, the divide symbol, and enter the value of 10 in the formula box. Define the new computed data set as something like **Size10_T6** in the *Result* box to document and click on the **Compute** command. This new data set will be used in calculating the finite element mesh resolution so that the element edges have a length of the order of this maximum size. The mesh will be denser where the size values are smaller.

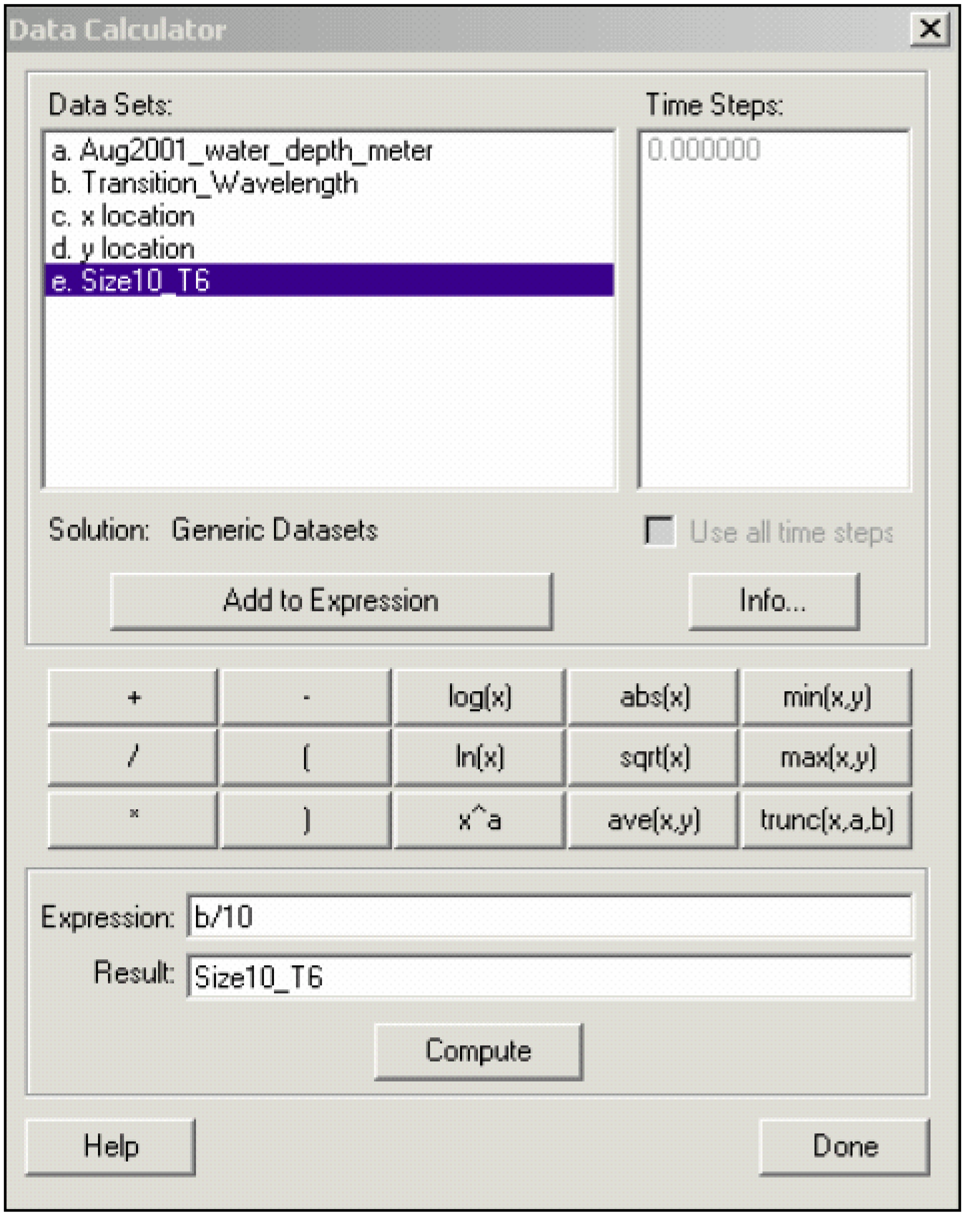


Figure A7. Create size function using data calculator.

Build polygons

The third step in generating finite element mesh is to build the polygons. Switch to the Map module and select the **Feature Objects|Build Polygons** command to construct polygons from the feature arcs. The **File|Get Info** command or the **Get Module Info** button on the toolbar will bring up a dialog that will tell you how many polygons were created. In this example, there should be three polygons: one for the ocean and two for the detached breakwaters.

Once you have created polygons, you can use the *Select Feature Polygons* tool to select individual polygons. Activate the *Select Feature Polygons* tool, and double click a polygon to bring up the *Polygon Attributes* dialog. This dialog allows you to set various options, which SMS will use to create the finite element mesh. If *Mesh Type* or *Bathymetry Type* is set to an option that is dependant on a scatter set, a **Scatter Options** button will appear below the combo box, allowing the user to select further options.

Ocean polygons. For the ocean polygon (Figure A8), set the *Mesh Type* to **Scalar Paving Density**. This tells SMS to generate a mesh with a density interpolated from a size scatter set. Click the **Scatter Options** button to open the *Interpolate* window. Select **Size10_T6** for the *Scatter Set to Interpolate From*, **Linear** for the *Interpolation Option*, and **Single Value** with a value of 0.0 for the *Extrapolation* box.

For the *Bathymetry Type*, input **Scatter Set** to indicate that the mesh elevations will be interpolated from scatter set data. Once again, a **Scatter Options** button will appear and selecting this button brings up a similar *Interpolation* dialog. The same input as before should be entered except that the elevation data set should be input for the *Scatter Set to Interpolate From*.

Finally, the *Polygon Type/Material* is input as **Ocean**.

Breakwater polygons. The other two polygons represent the detached breakwater polygons. The attributes for these polygons will have a *Mesh Type* of **None** and the *Polygon Type/Material* as **Land**.

Assign reflection coefficients

The fourth step is to assign the reflection coefficients. Values range from 0.0 for no reflection (i.e., complete transmission) to 1.0 for complete reflection. Typical reflection coefficients for marshy shorelines and rubble-mound breakwaters are 0.1 and 0.5, respectively. The public piers at Tedious Creek were constructed using vertical sheet-pile walls that did not extend to the bottom. The appropriate reflection coefficient of 0.9 was assigned for them. In the Map module, activate the *Select Feature Arc* tool and double click on each arc to set the appropriate reflection coefficient. This process is illustrated in Figure A9. Thompson et al. (1996) provides a good reference for typical reflection coefficients for short waves.

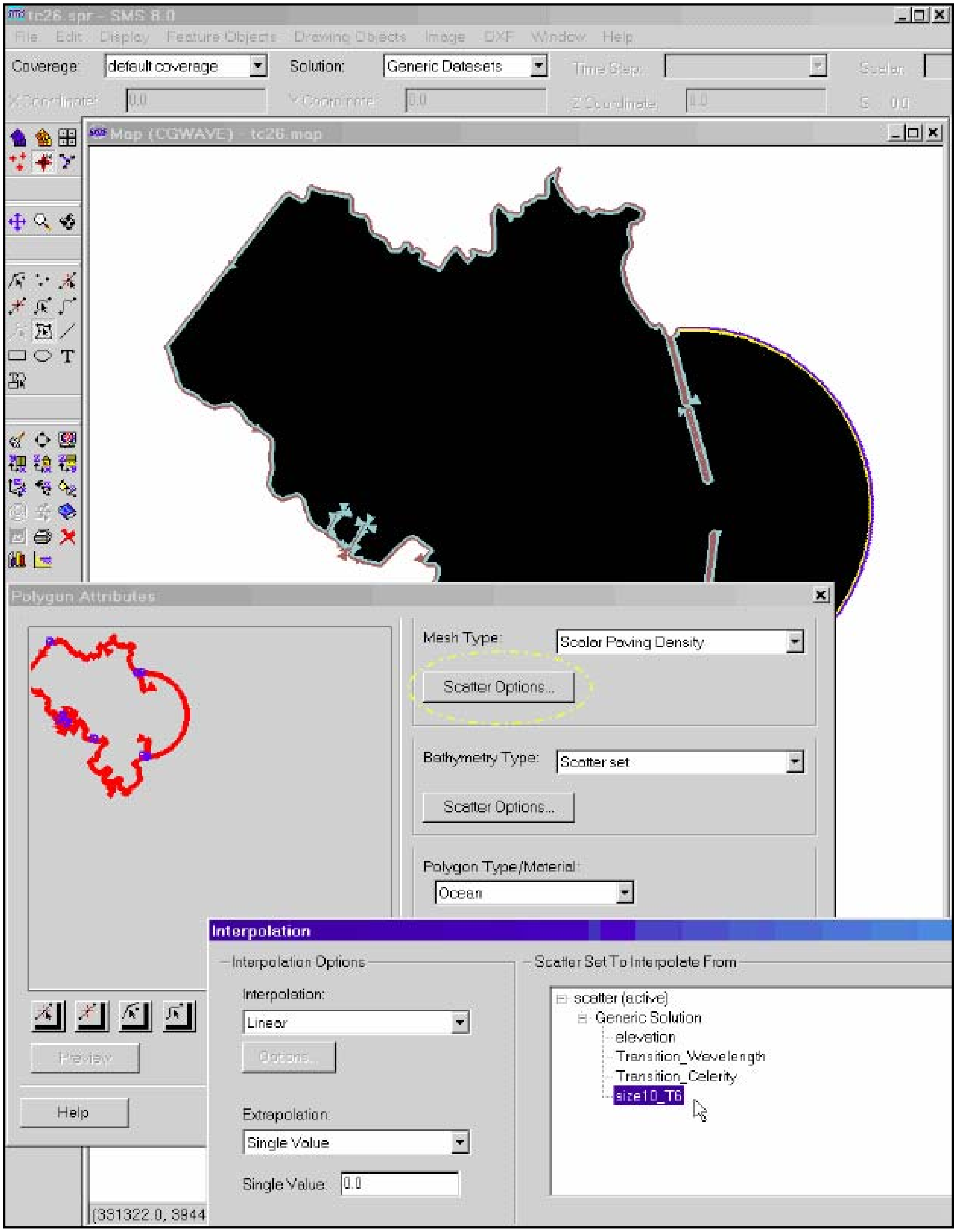


Figure A8. Defining attributes for polygon representing ocean material type.

Create mesh

This is potentially a long process, depending on the size function and the extent of your domain. It is a good idea to save your work first using the **File|Save** command. Then, select the **Feature Objects|Map to 2D Mesh** command to begin the mesh generation.

Figure A10 illustrates the sections of the created Tedious Creek CGWAVE mesh in the vicinity of the public piers and north breakwater. It illustrates the finite element mesh based on the T = 6-sec wave period and a size function defined by one tenth of the wavelength. The mesh contains a total of 339,042 elements and 171,265 nodes.

The primary purpose of this Tedious Creek study was to compare existing and authorized breakwater configurations. Figure A11 shows the two breakwater lengths relative to the channel outlines (red outlines). To accomplish this, a different mesh using the same size function definitions, with the revised arcs representing the altered breakwater alignments, was created.

Check mesh quality

Element and mesh quality can be checked using the **CGWAVE | Model Check** command in the *Mesh* module. A mesh should have certain properties to ensure that it runs efficiently during execution and does not cause instabilities in the solution. It should have (a) good elemental properties, (b) smooth bathymetric contours, (c) gradual area change, and (d) mild longitudinal depth changes. The elemental properties include aspect ratio, shape, and angle. An ideal element has an aspect ratio with sides that are the same length, no thin triangles, and interior angles greater than 10 deg. Adjacent elements should not have area or depth changes greater than 20 percent and should follow depth contours. If the *Model Check* does not return any serious errors, proceed to assign CGWAVE model parameters.

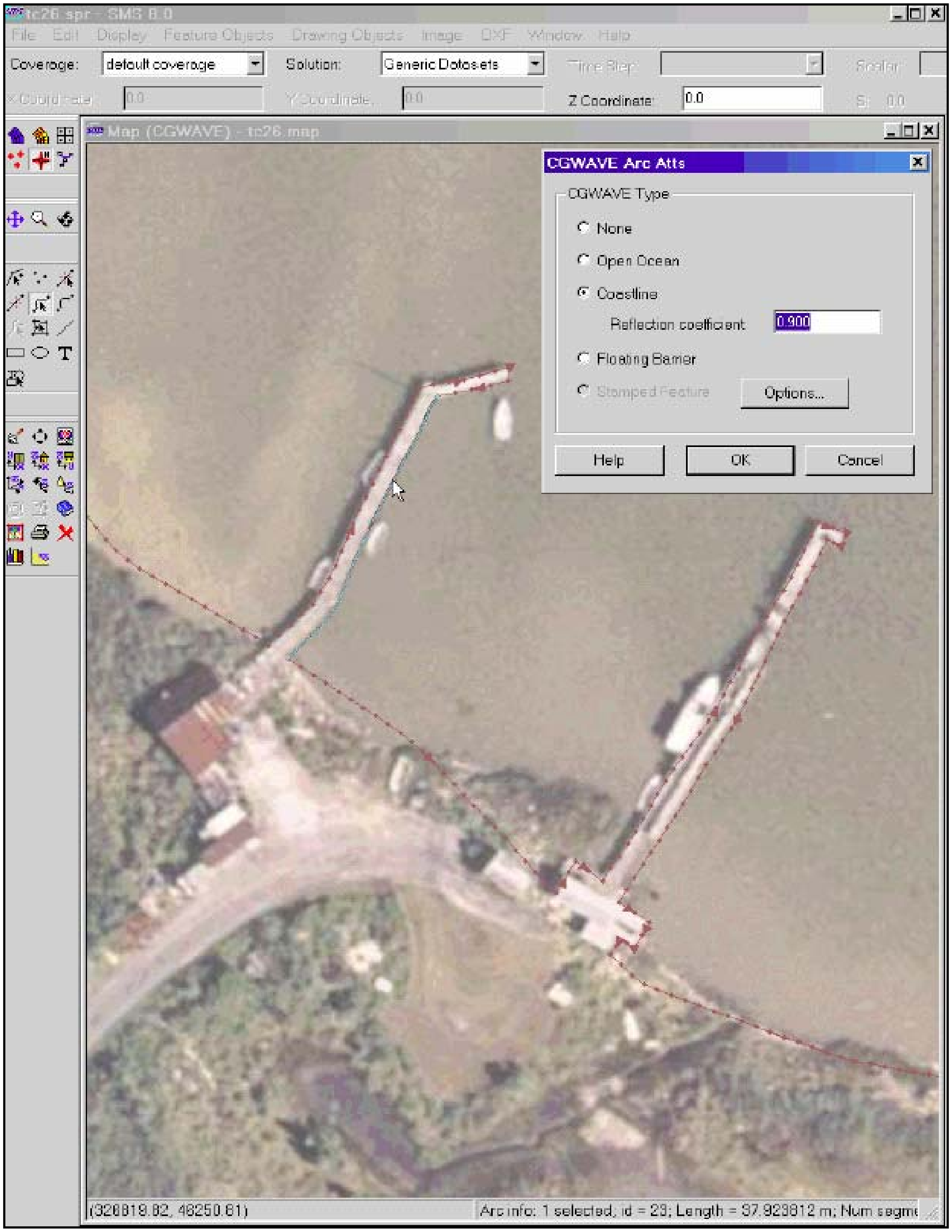
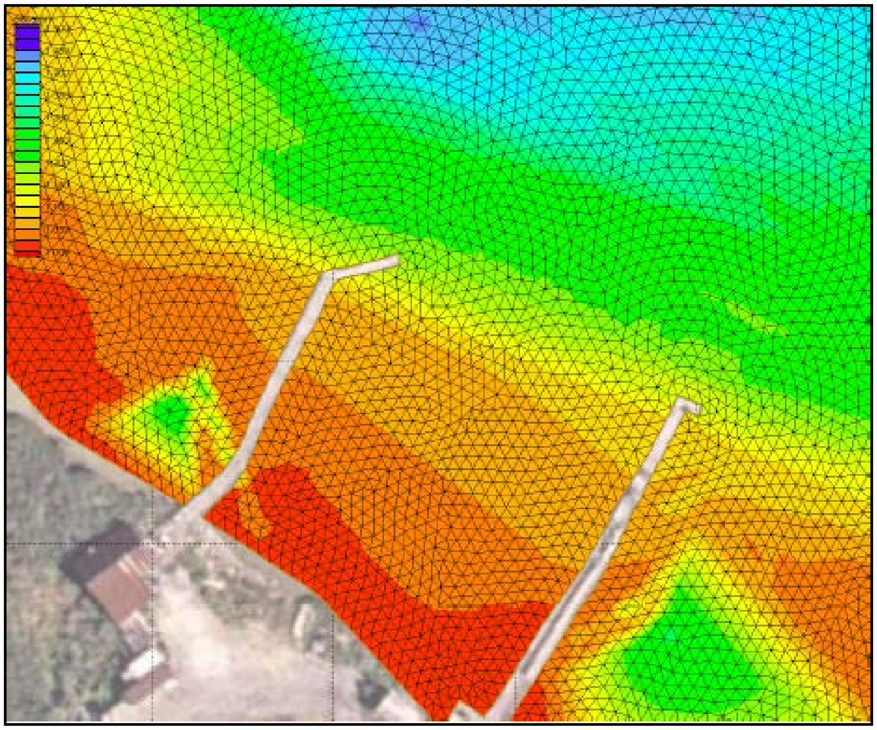
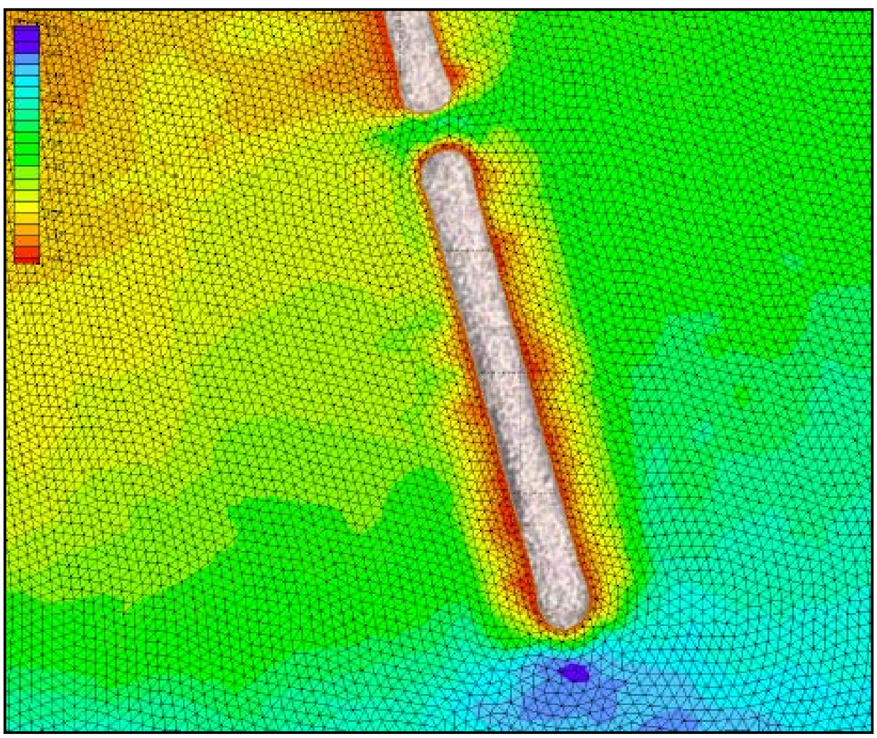


Figure A9. Assigning unique reflection coefficients to public piers.



a. Vicinity of public piers for existing conditions



b. Vicinity of north breakwater for existing conditions

Figure A10. CGWAVE mesh and bathymetry at Tideous Creek, MD.

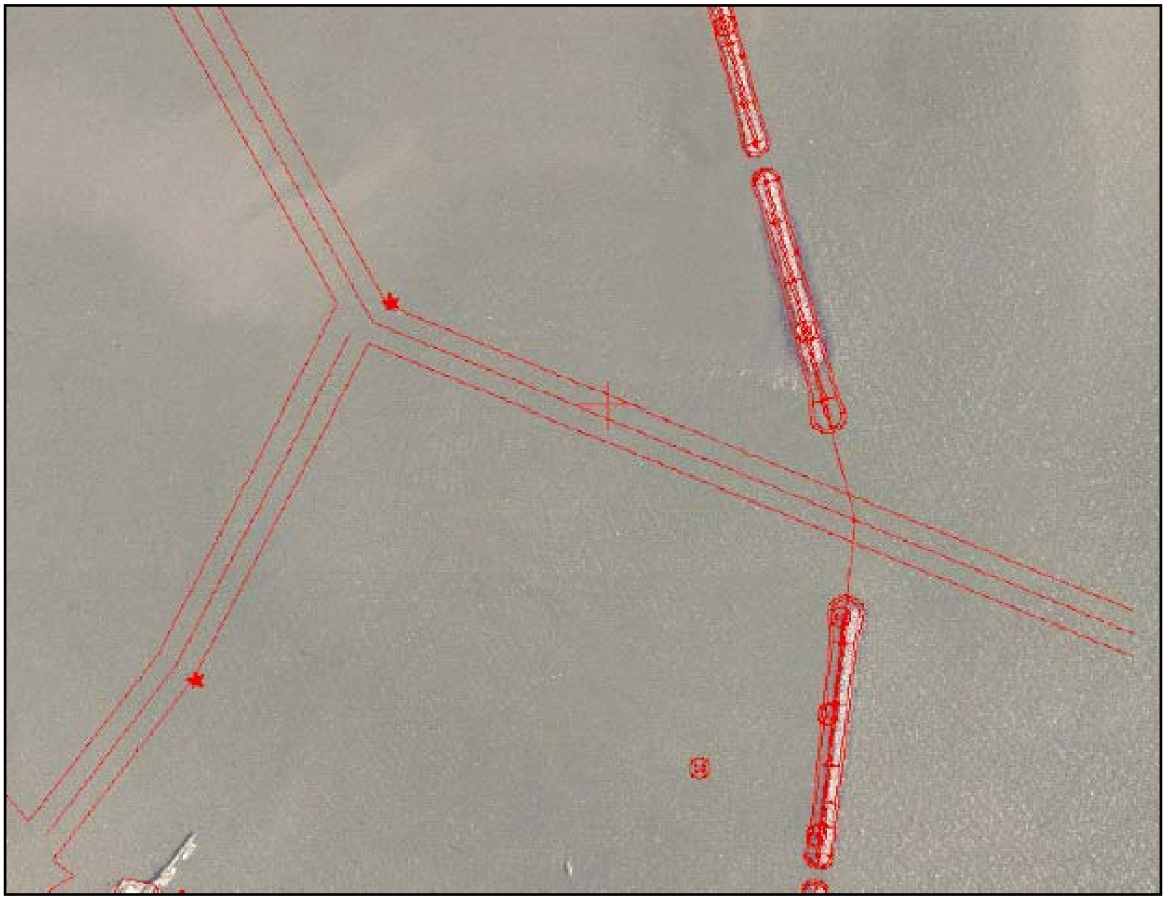


Figure A11. Schematic of existing breakwater configuration (121.9-m (400-ft) gap) versus authorized breakwater alignment (91.4-m (300-ft) gap) (red outline) with respect to channel outline, Tedious Creek, MD.

Assign model parameters

Defining the CGWAVE model parameters is a two-step process: (a) renumber nodestrings, and (b) input model control parameters.

Renumber nodestrings. Nodestrings are subsets of sequential nodes that allow specific parameter assignments and operations. SMS automatically creates nodestrings from the feature arcs when it generates the mesh. The nodestrings should retain the properties, such as reflection coefficients, that were already assigned to the feature arcs so all you need to do is renumber them. Renumbering gives new numbers to all the nodes in an orderly fashion to improve numerical bookkeeping. It only has to be done once after all mesh editing is finished. Typically, the best renumbering location is the ocean boundary nodestring. The mesh is renumbered by selecting the **Nodestrings | Renumber** command from the *Elements* menu after having selected a nodestring. The nodestring is

used as a seed to start the renumbering process. The bandwidth option is the default.

Input model control parameters. Input the model control parameters by selecting the **CGWAVE | Model Control** command in the *Mesh* module. These parameters include sections on (a) incident wave conditions, (b) nonlinear wave options, (c) open boundary, (d) 1D wave transformation inputs, and (e) numerical solver. Incident wave conditions are input along the open ocean boundary and include wave period, direction, and amplitude. The amplitude is one half the wave height and should not be confused with the wave height. Nonlinear wave options include bottom friction, wave breaking, and nonlinear dispersion. When first debugging a model, it is best to do the simple cases first. Then, a sensitivity analysis can be performed with these wave options to determine which should be exercised. Open boundary conditions should match those selected when generating the domain in the *Map* module. These are semicircular in the Tedious Creek example.

In the 1D section, the user can specify either the number of nodes or the spacing in the 1D lines. The spacing should be of the order of the smallest element, typically closest to the coastline, on the semicircle. A rule of thumb is that it is 1.25 times the offshore boundary radius divided by 100. The length of 1D lines should extend sufficiently offshore, and may extend to the limits of the existing scatter set or be further extended to pick up waves from buoys located beyond the scatter set. After selecting appropriate accuracy, the user selects either the *1-d nodes* or the *1-d spacing* in the Compute section to calculate and save the 1D lines (assuming the *Save 1-d file* under *Open Boundary* has been selected).

Finally, the numerical solver should be selected for the machine performing the calculations. If the model is small enough for the PC, then the “0 (or 1) Standard (PC)” solver(s) is selected. Otherwise, the “2 SGI Parallel” solver for the HPC Ruby SGI processor should be selected. There are different solvers for different HPC supercomputers, however.

Run model

Before running the model, save your file and make an additional backup copy with a different name in case the file becomes corrupted. Select **File | Save As** to save the file as a project file with suffix **.spr*. All geometry, input, and mesh information is contained in this file.

ERDC MSRC resources

CGWAVE can be run from within SMS, but may be "painfully" slow depending on the size of your mesh. To run on your PC, select **CGWAVE | Run CGWAVE**. To run on the HPC Ruby SGI processor, you must have (a) Kerberos software (latest Version 20030506b for Windows) to ensure secure communications between your PC and Ruby, (b) a SecurID card for obtaining a "ticket" to use the ERDC Major Shared Resource Center (MSRC) resources, (c) telnet software such as PuTTYtel (telnet) and (d) file transfer (i.e., FTP) capability, such as windows-based Filezilla. The latest version of the Kerberos software and telnet/ftp software can be downloaded from the MSRC Web site at *http://kirby.hpcmp.hpc.mil*. The *Documentation* tab on the left side of the screen is useful for details on the procedure. The ERDC MSRC Customer Assistance Hotline at 601-634-4400, option 1, is particularly helpful. Your CGWAVE files can then be copied to the Ruby (ASCII mode), executed, and then copied back to your PC for post-processing within SMS.

HPC Ruby access

The first step is to gain permission to use the MSRC resources. All examples in this appendix assume the UNIX 6.5 shell. For the UNIX environment, it is advisable to have filenames without embedded blanks. The University of Texas maintains a Web site with a good introduction to UNIX that is accessible at *http://www.utexas.edu/cc/unix/index.html*.

Initially, Kerberos and Filezilla software are used to copy two SMS output files from your PC to Ruby. These ASCII files are the CGWAVE run control input file (i.e., *.cgi) and the 1D (*.cg1) file. The asterisk is a placeholder for the project prefix selected for the study. After successful transfer of these files, a PuTTy telnet session to Ruby is required to issue UNIX-based commands. The two input file names need to be renamed. The first file *.cgi must be renamed to be *.dat and the *.cg1 file must be renamed to *.1d. This is accomplished with the copy UNIX command that follows the format cp oldfile newfile.

CGWAVE execution

There are four steps for executing CGWAVE within Ruby, including (a) edit input file *.dat, (b) convert input to binary, (c) run CGWAVE, and (d) convert output to ASCII.

The first step is to edit the *.dat file to reflect changes in incident wave period, amplitude, and wave direction in the title (for documentation) and the corresponding input field near the beginning of the file. Use the line editor “ed” or the full screen editor “vi” within the UNIX environment. A quick reference guide for the vi editor is *http://cac.uvi.edu/miscfaq/vi-cheat.html#toc*.

The second step is to convert the input ASCII run control file from SMS to binary format. The utility program “dat2unf” performs this conversion and creates three files, *.par, *.geo, and *.grd, corresponding to the input parameter, geometry, and grid files, respectively.

The third step is to run CGWAVE using the executable file pcgw_sgi_ser. This is the serial version for the SGI processor Ruby (i.e., same as option 2 under **CGWAVE | Model Control** described earlier). This is the main program that does the CGWAVE calculations. CGWAVE runs usually required less than 5 min to complete up to eight iterations. Thus, the serial version is adequate for the size of this model. Larger models would benefit from HPC’s parallelized codes and solvers.

The fourth step is to convert the binary output file *0001.res generated from the third step to ASCII using the utility program “res2out_sgi.” The output file *.out from this program can then be transferred in ASCII mode to your PC via Filezilla FTP. It is a good idea to copy this output file to your home directory on Ruby with a descriptive name and suffix .cgo before transferring to your PC. SMS recognizes this file extension better than the *.out suffix. The *.che file is a documentation file recorded during each CGWAVE run (i.e., the pcgw command) to show the resolution obtained after each iteration. It should also be saved and transferred to your PC for future reference.

The final step after each run is to purge the miscellaneous files in the Work directory before executing an additional run. The user can create a shortcut command in the alias file. For Tedious Creek, a shortcut command **rmtcall** was created to remove these files so that they could be reused. The **up arrow** command works for cycling through all commands executed during a telnet session on Ruby. This is helpful if multiple runs are planned, so that the commands do not have to be typed in again. After all runs, type **Exit** and carriage return twice to exit Ruby.

Post-process results

CGWAVE calculates scalar and vector output information. Scalar data include wave height, phase, sea surface, dynamic wave pressures at three depths, and wave surface data. Vector data include particle velocity at three depths, wave direction, and wave velocity. Usually, the wave height and phase are the primary scalar quantities, and wave direction the primary vector quantity of interest.

Results from the CGWAVE runs can be post-processed in SMS. The first step is to FTP the ASCII result files to your PC using the Filezilla software. The next step is to import the *.cgo data files in the **Data Browser | Import** command of SMS.

Visualization methods in SMS include contour plots, vector plots, animations, and plots of observation data. The display options tool can be used to quickly access contour and vector display options.

Contours

To view contours of scalar data within SMS, select the *2D Mesh* tab in the *Display Options* dialog and turn on the *Contours* toggle. The *Contour Options* tab controls how the contours will be displayed. Contours can be linear, filled, or combined. The range and either the number or size of intervals can be specified. When you exit the dialog, the display will update with contours corresponding to the active scalar data set.

Vectors

To view a vector plot within SMS, turn on the *Vectors* toggle on the *2D Mesh* tab of the *Contour Options* dialog. The *Vectors* tab allows you to specify head size, arrow length, colors, and placement of vectors. The vectors will be displayed according to the active vector data set.

Animations

Animations can be created within SMS from the time steps that are imported for each solution. A series of wave height contours or wave direction vector plots is formed to create the animation with the number of frames determined by the user. To create an animation, select the *Film Loop…*item from the *Data* menu in the Mesh module. An animation wizard will guide you through the process of creating a film loop. The film

loop will be saved as an .avi file that can be viewed from within SMS or used with other software such as Microsoft PowerPoint.

Observation plots

An observation coverage can be created to look at a cross section of wave heights along a user-defined transect. Data can be plotted within SMS and saved for further post-processing with other software (i.e., Axum, Excel, etc.).

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14. ABSTRACT

Tedious Creek is a small, funnel-shaped estuary located on the eastern shoreline of the Chesapeake Bay in Dorchester County, MD. Prior to the construction of the breakwater in 1997, the orientation of Tedious Creek allowed the transmission of storm waves that, at times, caused substantial damage to local vessels. The breakwater differed in geometry from the plans tested in 1994. Foundation problems encountered in the field resulted in a shortening of the breakwater, and a wider opening between the two breakwater sections resulted. Local watermen complained that the breakwater was not providing the authorized level of protection at the county boat dock and public piers on the south shore. It was suggested that the as-built 122-m (400-ft) gap opening should be reduced to the authorized 91-m (300-ft) gap opening.

The objective of the Monitoring Completed Navigation Projects study was to determine if the harbor and its structures were performing (both functionally and structurally) as predicted by model studies used in the project design. Specific field data would be obtained and analyzed. These data were used in numerical simulation modeling to ascertain the level of wave protection provided by the as-built breakwater structure, and to compare this level of protection to that which would have been provided if the authorized structure had been built. A third hypothetical structure with a 61-m (200-ft) gap opening also was evaluated. No adverse environmental effects such as breakwater deterioration, shoreline erosion, or scour near the breakwater could be ascertained by analyses of these data.

(Continued)

15. SUBJECT TERMS
See reverse.

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14. ABSTRACT (concluded)

Wave height transformations were performed with varying wave heights, tides, storm surge levels, and incident wave angles using numerical models STWAVE (no diffraction), STB3 (diffraction), and CGWAVE (refraction, diffraction, and energy losses). Evaluations were performed for (a) storm waves, as-built and authorized structures, (b) moderate waves, as-built and authorized structures, (c) typical daily waves, as-built and authorized structures, and (d) storm and typical daily waves, hypothetical structure. For all wave conditions, any reduction in wave heights at the county boat dock and public piers by reducing the gap opening in the breakwater from the as-built to the authorized opening would be minimal and insignificant. Reduction of the gap to the hypothetical 61-m (200-ft) opening resulted in about a 10-percent reduction in typical daily condition (considered insignificant) and modification of the structure to this degree (from as-built 122-m (400-ft) gap to hypothetical 61-m (200-ft)) would not be justified.

The functionality of circulation and flushing of the as-built condition was evaluated by applying two models (RMA2 and RMA4) within the TABS-MD suite of numerical models. RMA2 was used to demonstrate general hydrodynamic circulation patterns resulting from verification of the August 2001 field data. RMA4 was used to analyze harbor flushing. The as-built condition appears to maintain good harbor circulation, with velocities below any threat to boats that frequent the harbor. RMA4 flushing tests indicate that the harbor has adequate flushing, and compares favorably to the no-structure flushing test.

Moreover, field data and observations made by the Baltimore District during project location site visits indicate that wave conditions preventing satisfactory operations at the county boat dock facility often result from northwesterly waves generated locally on Tedious Creek, rather than by waves propagating through the breakwater gap from a northeasterly direction.

15. SUBJECT TERMS

Breakwater
CGWAVE wave modeling
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